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GUIDELINES FOR DESIGN, CONSTRUCTION AND MAINTENANCE OF CABLE STAYED BRIDGES

INDIAN ROADS CONGRESS

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INTRODUCTION

Latest advancements and rapid developments in construction materials and machinery, design philosophy, construction technologies, have changed the concept about how road bridges are to be planned, designed, constructed and maintained. Bridge design philosophy is a set of assumptions and procedures which are used to meet the conditions of serviceability, safety, economy and functionality of the structure. Several design philosophies have been introduced from different parts of the world. The working stress method used earlier has since given way to the Limit State Method (LSM).

In view of the recent focus on accelerated construction of large number of special bridge structures like Suspension bridges, Cable Stayed Bridges, Extradosed Bridges, Long Span Arch Bridges & Long Span Cantilever Bridges, need was felt to prepare separate documents for such bridges. Accordingly, Steel and Composite Structures Committee (B-5) undertook the work of preparing Guidelines for Cable-supported Bridges and Structures (Part A: Cable Stayed including Extra-dosed Bridges") during its tenure 2015-2017 and prepared initial draft. While deliberating on the comments offered by the then B-9 Committee on the draft guidelines, the B-5 Committee in its 10th Meeting held on 22nd September, 2017 opined that there is a requirement of Code rather than a Guidelines and recommended that B-9 Committee takes over the responsibility of preparing the document for Cable Stayed Bridges alongside the document they devising for the Extradosed Bridges as this will reduce conflict and also cut down on the technological input to be made for preparation of these two highly needed documents for the engineering fraternity.

Thus preparation of a document for Cable Stayed Bridges was taken up to cover concepts, planning, aesthetics, design, analysis, wind and seismic effects, material, quality control, bearings, inspection, maintenance, structural health monitoring, etc. The task was undertaken by B-9 Committee during the tenure 2015-2017 headed by Shri G Sharan which carried on for two successive tenures 2018-2020 and 2021-2023 for devising such an exhaustive Guidelines alongside Guidelines for Extradosed Bridges. For this purpose, a Sub Group B-9.4 was constituted under the Chairmanship of Prof. Mahesh Tandon comprising experts viz Shri G Sharan, Prof. Prem Krishna, Dr. B.P. Bagish, Dr. N.C. Pal, Dr. B.C. Roy, Dr. Lakshmy Parmeswaran, Dr. Harshavardhan Subbarao, Prof. S.K. Thakkar, Prof. Yogendra Singh, Dr. Suresh Kumaresan S/Shri Inki Choi, V.N. Heggade, Vinay Gupta, Alok Bhowmick, C. Sankaralingam, Mohan Jatkar, U.K. Rajeshirke, P.G. Venkatram, J.S. Pahuja, Nirav Mody and Jitendra Rathore. The Draft document was discussed in a number of meetings of the B9.4 Sub Group and B-9 Committee before it was finally approved by the B-9 Committee in its meeting held on 19th September, 2022 for placing before the BSS Committee for its consideration, and, for forwarding it to IRC Council. The list of personnel of B-9 Committee for tenure 2021-23 is as under:

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The document finalized by the B-9 Committee was approved by the Bridges Specifications & Standards Committee (BSS) in its meeting held on 25th September, 2022 and subsequently, the Executive Committee approved the same in its meeting held on 7th October, 2022 and authorized Secretary General, IRC to place the same before the Council. The "Guidelines for Design, Construction and Maintenance of Cable Stayed Bridges" was approved by the IRC Council in its Annual Meeting held on 8th October, 2022 at Lucknow (U.P.) for printing.

The cable-stayed bridge form is eminently suited to spans in the range of 200 - 1000 m. The current record is held by the Russky Bridge in Vladivostok, Russia, with a main span of 1104 m., In Annexure A1, data of bridges in India, where available, has been summarised in a Table for ready reference. The cable stayed bridge form consists of a light slender deck supported by closely spaced stay cables which carry part of the load of the deck to the towers (pylons) that transfer the forces to the ground. The balance of the load is transferred to the pylons through flexure of the deck. The high structural efficiency of the form is derived from the light slender deck, the stay cables in tension, and the towers that transfer the cable forces to the foundations through vertical compression. The tensile forces in the stay cables also cause compression in the deck, often considered as 'free prestressing'. Popularity of cable stayed bridges is also on account of the fact that enormous possibilities of aesthetics are offered by the form itself.

Slenderness of the deck has its complications in design because of the excitation of higher modes of vibration and the coupling thereof. While the light weight of the deck is of some advantage from seismic considerations, the opposite is the case for aerodynamic considerations. The main disadvantage of the cable stayed form is that the construction stage configuration is often more onerous than the service stage for transverse wind and it is not unusual to look for temporary stabilizing arrangements including dampers for long spans.

Chapter 1 of the Guidelines defines the scope in terms of maximum span length and the materials used for deck construction. The scope of these Guidelines shall be applicable for cable stayed bridges with maximum main span lengths as follows:

Concrete…........................upto 350 m

Composite and Steel……..upto 500 m

The Design Life of the Cable-Stayed Bridges will be taken as 100 years to be in sync with other current IRC codes.

Chapter 2 introduces the modern Cable-Stayed Bridge Construction with a brief history of its evolution. The typical features along with the flow of forces in the pylon, deck, and stay cables and the advantages and disadvantages of cable-stayed bridges are discussed.

Chapter 3 gives a brief overview of the structural forms of cable stayed bridge system and its three main components, i.e., the deck, the tower (pylon), and the inclined stay cables. The interaction of these components makes the structural behaviour of each cable- stayed bridge unique and fairly complex.

Chapter 4 describes the structural action of Cable-Stayed Bridges. This chapter explains various structural concepts through bridges that have been constructed in India.

The most suitable material for the decks of different main span ranges for Cable Stayed Bridges has been identified by Svensson **[1]** as follows:

Concrete……...< 400 m

Composite…… 400 - 600 m

Steel……… .....> 600 m

Chapter 5 explains the construction methodologies adopted for cable stayed bridges. The cable-stayed bridges differ from conventional girder bridges, like use of cables, construction of upper pylon, installation and anchoring of cables, etc. The stresses and deformations produced in the members can be significantly influenced by the construction method and sequence of construction.

With regards to construction, it is necessary to satisfy required conditions such as environmental effects, construction safety, cost and period, which are based on its relation to entirety, purpose, type, scale and characteristics of project.

Prior construction planning is necessary for performance requirement of the project, in addition to the aspects of the quality and schedule control

As the influence of construction method, construction sequence and construction accuracy are significant in cable stayed bridges, chapter emphasizes on the importance of stationing the construction management team at site with necessary experience & sufficient knowledge.

Chapter 6 deals with Loads and Load Combinations to be considered for the cable stayed bridge. In general, the provisions of relevant codes for Concrete Bridges (IRC:112), Steel Bridges (IRC:21) and Steel- Concrete Composite Bridges (IRC:22) are applicable for structural design.

The weight of stay cables to be considered in the analysis and fatigue considerations has been specifically covered in this chapter since this does not find place in the above-mentioned codes. Similarly, the temperature difference between the stay cables and other parts of the bridge are given since the cables are more sensitive to temperature changes. The temperature difference between opposite sides of the pylon also must be considered in design for which guidelines have been specified.

The Accidental breakage of a stay cable must be considered as the sudden rupture of a highly stressed cable can result in dynamic augmentation of the force in the cable. The same is covered in this chapter.

The design life of the stay cable is much lower than that of the bridge as discussed in Chapter 10. This means that the stay cables would have to be replaced during the lifetime of the substructure, and this condition is to be considered for design. The allowable bending in the stay cable due to variable loads based on Fib 89 is covered here.

Although the Load Combinations for SLS and ULS checks must be carried out in accordance with IRC:6, the load factors and load combinations applicable for the Stay Cables have been tabulated separately in this chapter.

Chapter 7 of this guideline covers the analysis methods and design requirements for the design of cable stayed bridges.

Section-I, covers the structural modelling aspects for cable stayed bridges, including method to include the geometric nonlinearity of stay cables. Methods of analysis to obtain Second order effects in the pylon and deck which are slender elements are also covered in this chapter. The analysis method to be adopted for various limit states are explained. Finally, this section explains the methods to determine initial shape of bridge, obtain cable force (by preliminary calculations) and various construction stage analysis techniques which can be adopted during design stage of the cable stayed bridges.

Section-II of this chapter covers the design requirements of the stay cables for cable stayed bridges for Ultimate limit state, fatigue limit state and serviceability limit state. The fatigue consideration depending on the Wohlers Curve of stay cable is covered in this chapter. Based on Fib 89, the allowable stresses under SLS conditions shall be limited to 0.5 UTS, which shall form the basis of design. For ULS conditions, the strength shall be checked for UTS of the stay cable with a material factor of safety of 1.35.

In **Chapter 8** seismic effects have been examined. Cable stayed bridges are typically long span and flexible structures, requiring some special considerations for seismic design. Being torsionally flexible and having coupled modes of vibration, a 3D modelling and analysis becomes necessary, considering adequate number of modes of vibration. In case, it is not possible to achieve 90% mass participation in modal analysis, a time history analysis may be required. These structures are designed to remain elastic (R=1) for the design basis earthquake (return period 500 years) with applicable importance factor. Having long spans and long lengths, cable stayed bridges are subjected to variation of ground motion underneath the different pylons/piers having coupled movement at the top due to continuity of the superstructure. These aspects have also been given an exposure.

In **Chapter 9** an effort has been made, to guide the reader in understanding the basics of Action and its Effects on Cable-Stayed Bridges. The various kinds of wind-induced oscillations (both limited and divergent amplitude) that can occur in cable-stayed bridge components, such as the deck, pylon, and cables, are briefly described. Also given is the methodology for checking the susceptibility of a cable-stayed bridge for wind-induced oscillations and to suggest the type of wind tunnel studies warranted.

Chapter 10 deals with Stay Cable Systems, Maintenance and Quality Assurance, covering in some detail, the corrosion & fretting protection requirements for the anchorages in transition zone and in the free length of the stay cables. Also included are levels of protection for preventing corrosion (internal barrier and external barrier) for various types of stay cable systems which are prevalent in newly constructed cable stayed bridges. Protection of stay cables from fire and vandalism are also emphasized. Various types of saddle systems used either for anchoring or passing of cable stay at pylon head with pros & cons for each saddle system have been included.

To mitigate vibrations in cable stays due to various effects as discussed in Chapter 9, different type of damping systems such as internal dampers, installed in connection with the cable anchorage and external dampers, installed independently of the anchorage and fixed to the deck are elaborated in this chapter.

Chapter 11 summarises the large variety of Bearings and Expansion Joints that can be employed in Cable Stayed Bridges. Given the length of continuous superstructure and the flexibility of the deck, movements at bearing and expansion joint can be as large as 1 m to 2 m. The bearings have often to be designed for rotations in the range of 0.05 – 0.15 radians. Hence, a highly durable and robust design and manufacturing of these devices becomes necessary and must be obtained from reputed manufacturers only. Apart from elastomeric, POT and Spherical bearings, Anti-Uplift Bearings are sometimes employed. If uplift is caused even under DL, it is usual to provide special devices which are mentioned in this Chapter. Since the design life of the bearings and expansion joints is much shorter than that of the bridge, arrangements for their replacement have to be incorporated. Modular expansion joints with full seismic movement capacity, seismic isolators, as well as devices like dampers of various types for different functionalities and STUs have also been discussed.

Chapter 12 deals with Geometry control which is an essential part of the construction monitoring process to ensure that finally the structure is translated in reality to the designed profile and alignment. The construction of cable stayed bridges, which involves stage construction of a flexible system, requires that the stresses, deformations in the structure and stay cable forces meet design expectations during the construction as well as during service. The geometric control procedures for different types of bridges and kinds of construction methodologies differ. This chapter gives guidelines for casting curves during concrete casting, procedure during fabrication and trial assembly for the segments and also during erection of the deck.

Chapter 13 deals with Inspection, Maintenance and Structural Health Monitoring of cable stayed bridges. The various parameters to be monitored, identification of critical location, sensors, data acquisition system, data transmission and management have been discussed with case studies. For general bridge structure, the structural monitoring contents consist of four parts during the bridge operation, i.e. environmental monitoring around the bridge, loading monitoring, structural response monitoring under different loadings, and durability inspection monitoring.

Chapter 14 makes an attempt to highlight some of the features of cable stayed bridges that enhance its aesthetics to a level that cannot be surpassed by ordinary bridges. All the members are predominantly subjected to axial forces, with the pylon and the deck under compression and the cables under tension which leads to a very efficient structural system. The flow of forces in the structure are self-evident and that is part of the enchanting appearance of this concept of bridge construction. The additional elements of pylons and stay cables introduced in the structure have a huge visual impact. The size of the pylon with its enormous height becomes the most dominating feature of the bridge, and, most of the time even the landscape. There are Cable stayed bridges that have been built in the urban environment not necessarily because of requirement of a long span but to create a landmark structure. Lighting of the cable-stayed bridge at night always forms an integral part of the design.

CHAPTER - 1

SCOPE

1.1 The scope of this Guideline covers road cable stayed bridges of the following types:

- Steel Bridges
- Steel Concrete Composite Bridges
- Concrete bridges – Cast in place or Precast

These Guidelines include analysis, design, construction, testing, monitoring and maintenance aspects of cable stayed bridges. The foundations and abutments of these bridges shall be designed and constructed in accordance with well-established principles as per other relevant IRC publications.

This Guideline shall be applicable for cable stayed bridges with maximum main span length up to 350 m with concrete decks and 500 m with steel-concrete composite or steel decks, though certain aspects of longer spans are also addressed briefly.

This Guideline intends to help the designer to evolve concepts that would lead to

- A safe and technically sound design,
- A design that permits a faster construction time and reduced material consumption.
- A safe and economical erection method,
- Low maintenance with the required service life,
- An aesthetically pleasing form,
- A low impact on the environment

1.2 Base Document

Cable Bridges are the subject of several Guidelines such as FIB 89 **[1]**, PTI **[2],** SETRA **[3]** and JRA (Japanese Specifications) **[4]**. These Guidelines have been formulated taking FIB 89 as the base document, but using the stipulations of Guidelines and Standards of the IRC and the BIS, to the extent available and applicable.

1.3 Design Life

The Design Life of the Cable Stayed Bridges will be taken as 100 years to be in sync with other IRC codes. However, in case it is desired to have a greater Design Life of the Cable Stayed Bridge, additional considerations come into play such as those enumerated below.

- Durability
- Discharge (and Scour)
- Seismic Effects
- Wind Effects

CHAPTER - 2

GENERAL

2.1 Overview

The development of cable stayed bridges can be said to have begun with the construction of a bridge in Germany by Loscher in 1784. However, the principle of supporting a beam or a mast by taught cables goes far back in time. Early examples are bridges from natural materials such as bamboo for the beam and Lianas for ties. The masts of sailing vessels were always transversely stayed by shrouds. By the time of World War II, the era of modern cable-stayed bridge design was just about to commence. The first modern cable-stayed bridge was the Stromsund Bridge designed by F. Dischinger built in the 1950's in Sweden with a main span of 183 m and two symmetrical back spans of 75 m each. More than 1000 cable stayed bridges have now been built worldwide. A significant number of cable-stayed railway bridges have also been constructed. More recently very elegant pedestrian cable-stayed structures have been built and have become landmark structures. Today, cable-stayed bridges have firmly established their unrivalled position as efficient and cost - effective structural form in the 200 m to 1100 m span range and even more. The cost effectiveness and the general satisfaction with their aesthetic aspects have propelled this span range in either direction, with both increasingly shorter and increasingly longer spans being designed and constructed at the present time. Usual spans of CSBs are larger than those of extradosed bridges and shorter than suspension bridges for the reasons of structural design & economy, with overlaps either side. Cable-Stayed Bridges (CSB) have become increasingly popular in India over the last 40 years.

A cable stayed bridge consists of one or more tall columns (normally referred to as pylon or pylons, also called tower or towers), with cables supporting the bridge deck as shown in **Fig. 2.1 (d).** In the cable-stayed bridge, the pylons form the primary vertical load-bearing structure. In many cases there are no vertical load carrying bearings to support the deck at pylon location and the deck is suspended only through stay cables whilst in other cases vertical load carrying bearings are provided. However, the nature of support in the longitudinal and transverse directions at pylon and abutment and /or hold down piers can vary to suite design requirements and site constraints. The early engineering approach, up to World War II, for the stay cables was essentially derived and hybridized from the already established engineering experience with suspension cables and post-tensioning technology.

In a regular cable stayed bridge the main span is the longer portion between successive pylons and the back spans (anchor spans) are the portion between pylon and abutment/hold down or anchor piers. Over a period of years, the arrangement of cables vis-à-vis their spacing along the span has undergone noticeable change. The Stromsund Bridge had only two cables on each side of the pylon, anchored to steel I-girders, which represents the trend in the 1950s. One of the reasons may have been the need to limit the degree of indeterminacy, and thus limit the computational effort required (modern computers had just begun to develop at that time). This required deeper girders to take up the bending moments generated. The modern trend is to have closely spaced (6 - 12 m) cables, thus reducing the bending moments, depth of the girders as well as the size of the cables. This is one of the reasons for the range of cable stayed bridge spans having becoming longer.

Cable stayed bridges have evolved over time and one could say that we are in the fourth generation of cable stayed bridges today. The earlier CSBs of post world war 1950s are said to be the so called first generation. Over the years there have been considerable changes in the cable strengths i.e. the ultimate tensile strength, the development of parallel strands/wires technology, better understanding of the aerodynamic behavior, vast improvement in wind tunnel and CFD studies and a quantum leap in computing power, computer technology and analytical design software, improvements in construction and erection technology, and use of high strength concrete, Ultra High Performance Fiber Reinforced Concrete (UHPFRC), high strength steel – have all lead to the increase in span length over time, be it a concrete, steel composite or steel deck. **Fig. 2.1 (a), (b) & (c)** shows the evolution of span length over time for steel, concrete and steel composite cable stayed bridges.

Steel cable-stayed bridges	Record span length	Year completed
Rhine River Bridge Düsseldorf North	260 m	1957
Severins Bridge	301 m	1959
Knie Bridge	319 m	1969
Rhine River Bridge Duisburg-Neuenkamp	350 m	1970
St. Nazaire Bridge	404 m	1975
Normandy Bridge	856 m	1995
Tatara Bridge	890 m	1999
Stonecutters Bridge	1.018 m	2009
Sutong Bridge	1.088 m	2008
Russki Bridge	1.104 m	2012

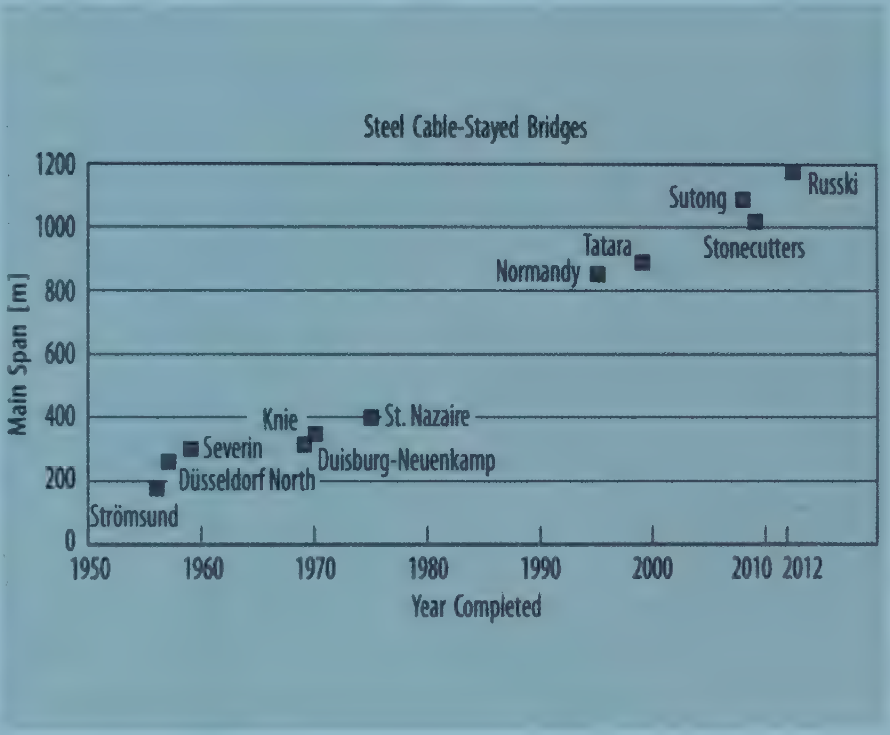


Fig. 2.1 (a) Record spans of Steel Cable Stayed Bridges

Concrete cable-stayed bridges	Main span	Completed
Maracaibo Bridge	235 m	1962
Wadi Kuf Bridge	282 m	1972
Brotonne Bridge	320 m	1977
Barrios de Luna Bridge	440 m	1983
Skarnsundet Bridge	530 m	1991

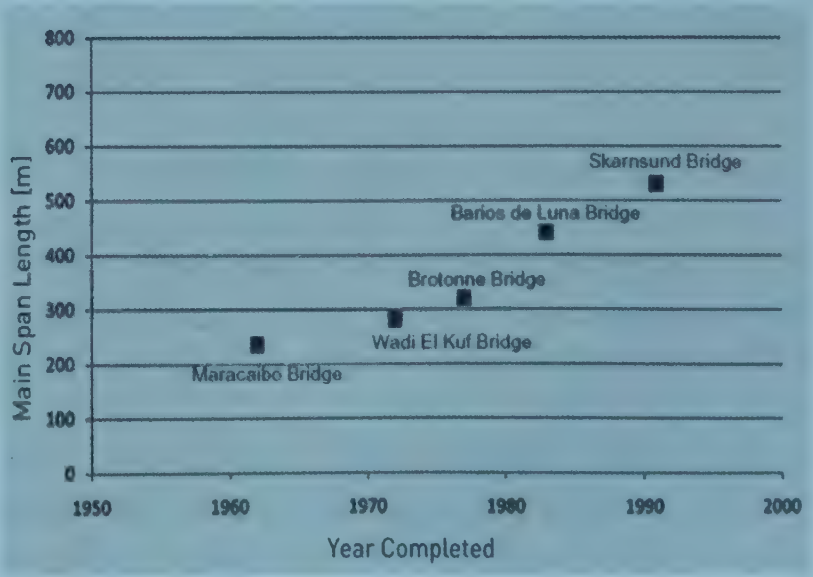
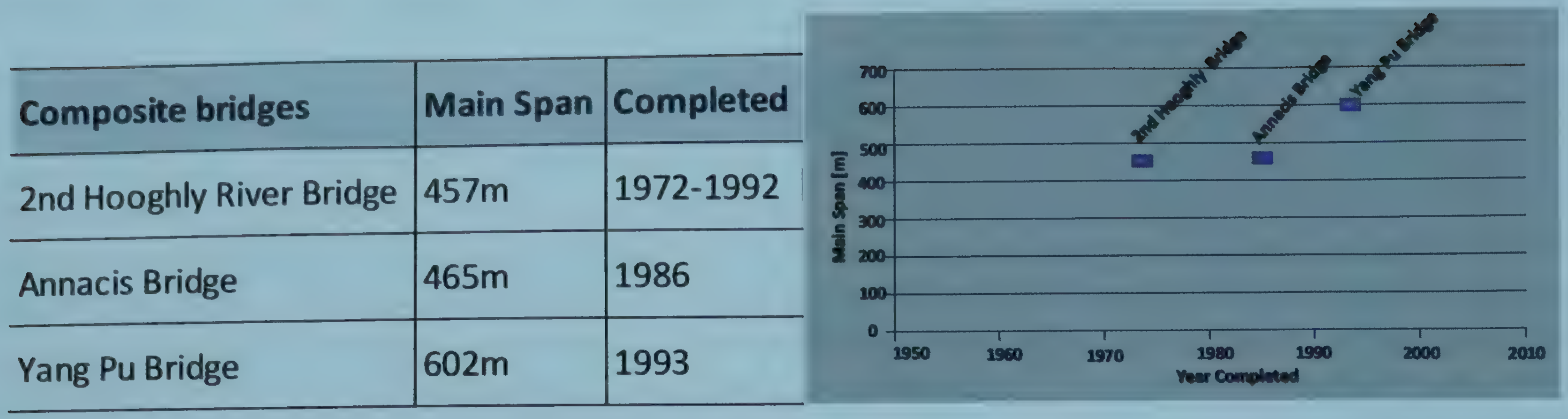


Fig. 2.1 (b) Record spans of Concrete Cable Stayed Bridges



Composite bridges	Main Span	Completed
2nd Hooghly River Bridge	457m	1972-1992
Annacis Bridge	465m	1986
Yang Pu Bridge	602m	1993

Fig. 2.1 (c) Record spans of Steel Composite Cable Stayed Bridges

The deck, as mentioned already, could be in reinforced or prestressed concrete, concrete-steel composite or in steel with an orthotropic deck. The choice would depend upon location, availability of material, labour and fabrication facilities.

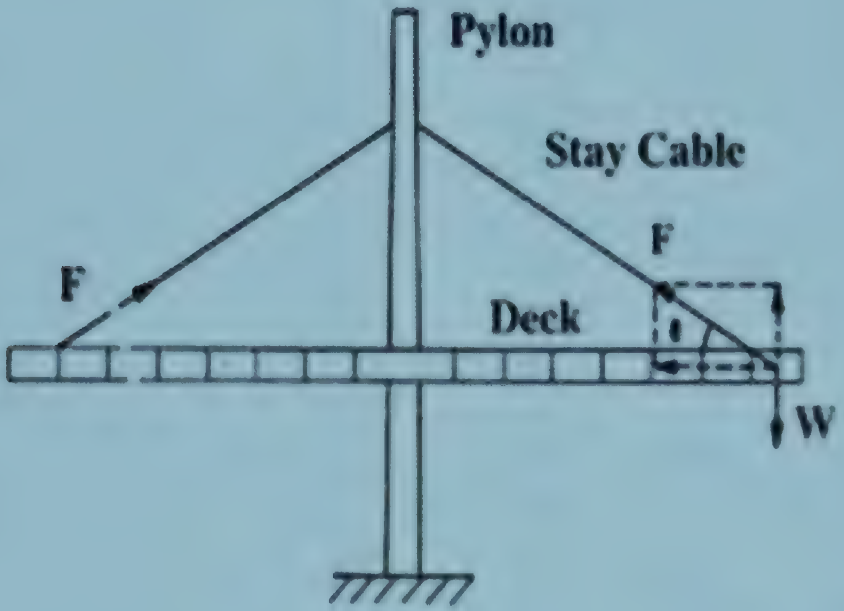


Fig. 2.1 (d) Triangle Structure: Deck-Pylon-Stay Cable [1]

The latest recommendations published by *fib* for stay cable systems **[2]** cover a significant number of new developments and specifications which forms the basis for this document and includes:

- Corrosion protection philosophy with a multi-barrier approach for the prestressing steel used for the stay cables and a rational design approach for structural steel components. A leak tightness test is also specified to verify the connection details between the free length and the anchorage zone of the stay cable and it is required at both ends of the stay and free length.

- Design and testing of stay cables for the inevitable flexural effects which occur close to the anchorages and other deviation points of the stay cables.

- Selected information on stay cable vibrations and special damping devices to control such vibrations.

- Suitable details for lightning protection of cable-stayed structures.
- Design consideration and testing procedures for stay cable saddles which are increasingly popular for cable-stayed structures with very slender pylons.
- State-of-the-art specifications for the main materials and components for stay cable systems including quality control procedures.
- Specific requirements for the common installation methods of stay cable are provided including the strand by strand installation and stressing methods.
- A comprehensive list of reference, relevant standards, and extended literature.

Post-Tensioning Institute, USA **[3]** and recommendations by the French Highway Administration, SETRA **[4]** have updated provisions on cable stayed bridges.

2.2 Features of CSB, Advantages and Limitations

The structure is made of three main components: Pylon, Deck, and Stay Cables. It can be compared to a triangular truss as in **Fig. 2.1 (d),** with the deck acting as the bottom chord. The stay cables are stressed at the time of erection in order to balance the weight of the deck. For subsequent loading conditions, the stay cables act as structural members as do the pylon and the deck.

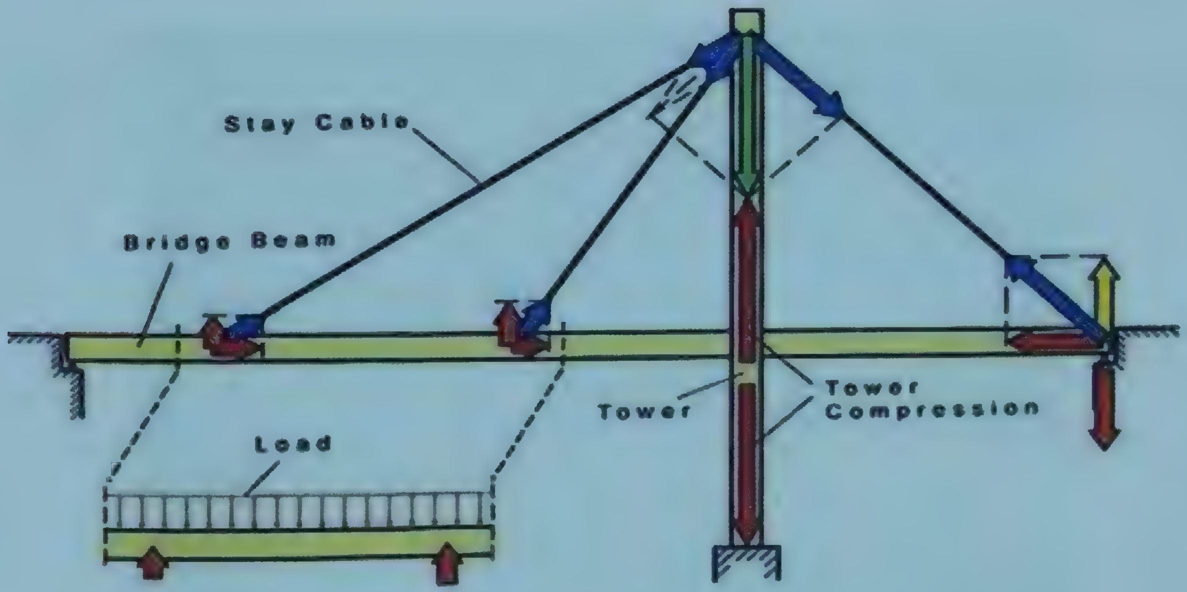


Fig. 2.2 Flow of Forces in CSB

Modern cable-stayed bridges have stay cables with well-defined and tuned cable forces which transfer their loads directly, **Fig. 2.2.** The horizontal components of the cable forces are introduced as compression forces into the deck. The forestay cables of the main span are tied back through the back stay cables to the ends of the bridge where these are anchored into hold-down (or anchor) piers. At the anchor point, each cable force is split into two components, the vertical force which is anchored to the ground and the horizontal force which is transmitted into the deck as a compressive force. The compression forces in the deck from the forestays and the backstays reach their maximum at the tower location.

CSBs offer a distinct advantage of being light yet stable, having shallow decks, aesthetically pleasing forms and are economical for spans of say, 200 m to 1100 m. However, while the central main span deck is suspended on one side of the pylon, the other side (the back span) cables are required to transmit the cable forces to the back span and balance the permanent loads to ensure overall structural stability. The longest back span cable is usually directly connected at the abutment and plays a vital role in the load balancing. Invariably, the back span is about 30% to 40% of the length of the main span in a regular CSB.

In case of Tatara Bridge, end span length was restricted due to existence of hills, hence, these were made in concrete, while the main span is in steel, in order to balance out the suspended load on both sides of the pylon. In case any of these features are not possible and the whole structure has to comprise of only one large span, then CSB is either not feasible or it requires special features like back stay anchored to ground (like Erasmus bridge in Holland, **Fig. 2.3**. Otherwise, an arch bridge or any other suitable form may have to be considered. Similarly, if a bridge/flyover with a large main span has to be located in the middle of a city, a CSB is not a preferred option from an aesthetical point of view as the pylon would cause a visual break in the urban habitat. However, these may be considered for ROBs where vertical clearance above rail tracks and matching the vertical profile to existing city roads are a constraint. In such cases, an Extradosed bridge may be more suited, instead of a CSB. In both cases, the depth of the deck is smaller than other conventional bridge types.

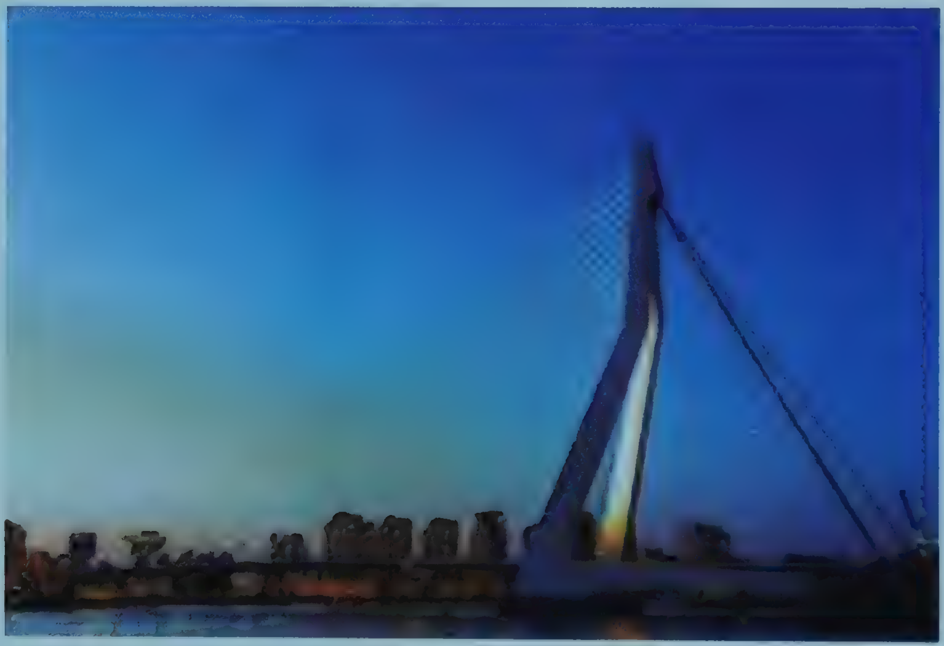


Fig. 2.3 Erasmus Bridge, Rotterdam, Holland

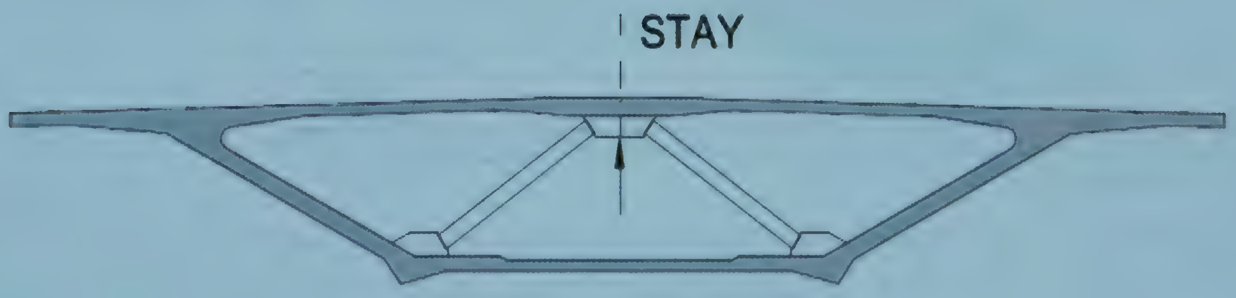
There are two main categories of modern concrete cable-stayed bridges:

1) **"Rigid" superstructure:** Box girders belong to this category. The box girder is used when high torsional rigidity is required: for instance, when the structure is supported with a central single plane of stay cables; **Fig. 2.4 (a).** Box girders may also be used to improve the aerodynamic stability of the bridge.

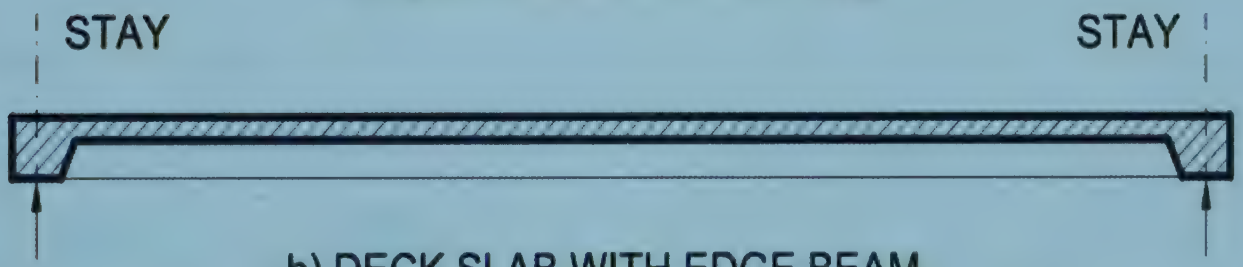
2) **"Flexible" superstructure:** This type of deck (with longitudinal edge girders as in **Fig. 2.4 (b)**, may be used when two or more planes of stay cables **Fig. 2.5** and **Fig. 2.6** support the superstructure and provide resistance to torsion. In Badhman CSB, three planes of cables have been used as in **Fig. 2.7**.

   The ratio of inertia between these two types of structures can be as much as 1/10.

   The flexible deck will more easily accept imposed deflections but will be more sensitive to buckling instability under high compression loads. The dead load of these two structural types is usually not much different. Longer steel box or steel-composite decks are lighter and more flexible than similar cross section types in concrete decks.



a) BOX SUPERSTRUCTURE



b) DECK SLAB WITH EDGE BEAM

Fig. 2.4 Deck Girder

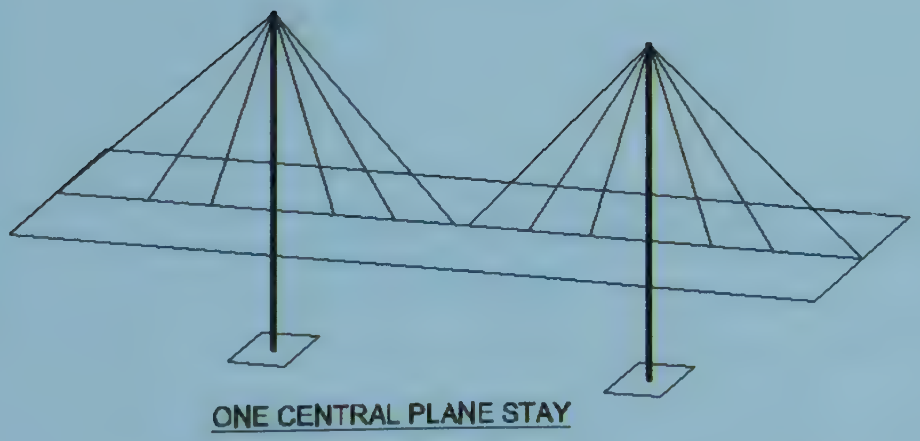


Fig. 2.5 Single Central Plane of Cables

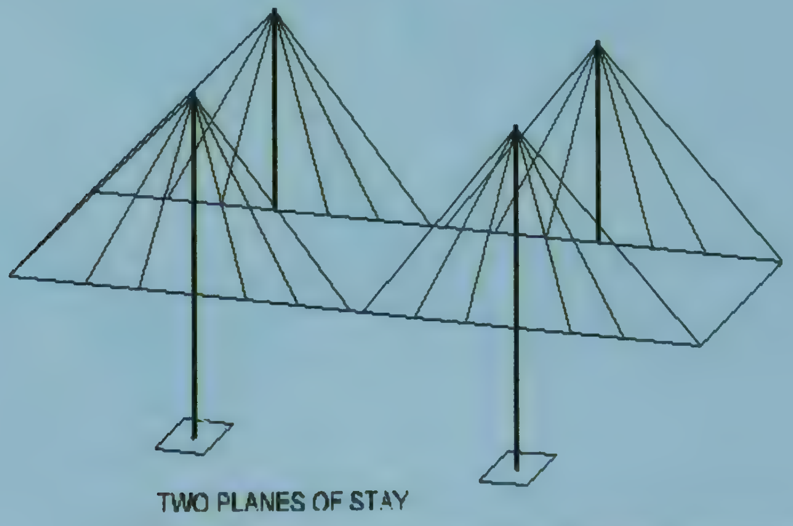


Fig. 2.6 Two Planes of Stay Cables

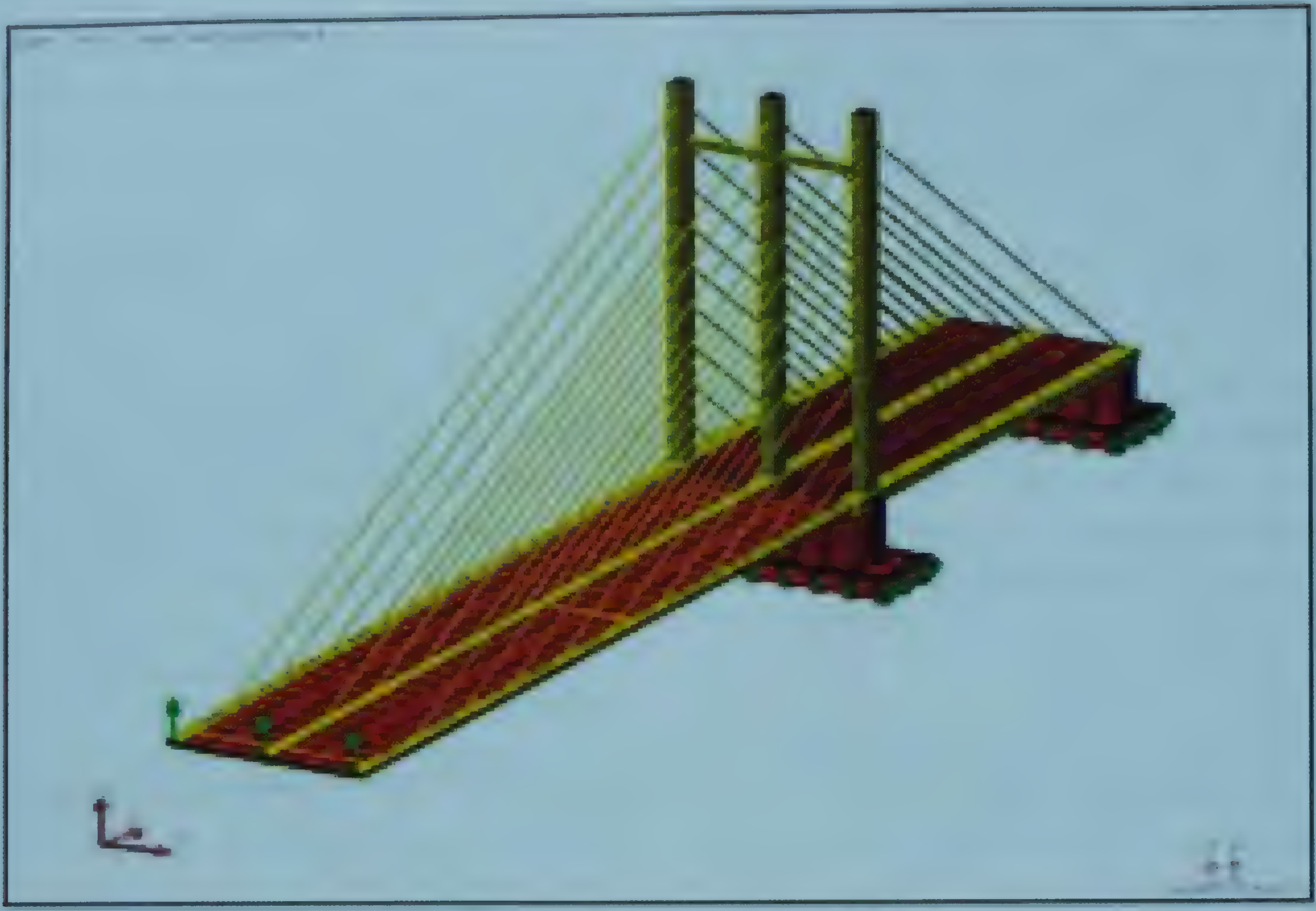


Fig. 2.7 Multi-plane Cables

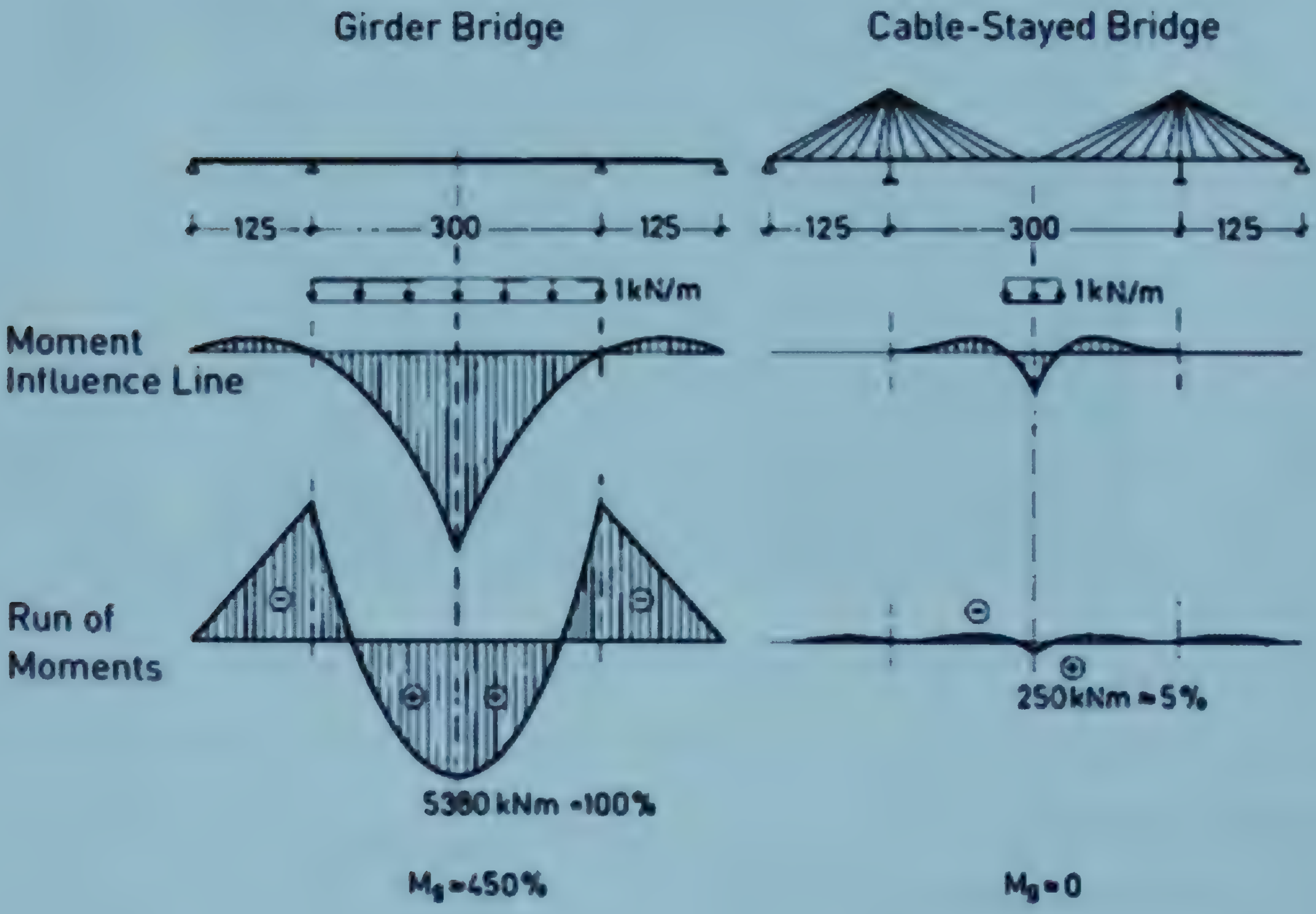

Girder Bridge
Cable-Stayed Bridge
125
300
125
125
300
125
1kN/m
1kN/m
Moment Influence Line
Run of Moments
⊖
⊖
⊖
⊖
⊖
⊕
5380 kNm =100%
250kNm ≈ 5%
Mg=450%
Mg=0

Fig. 2.8 Comparison of Beam Moments [1]

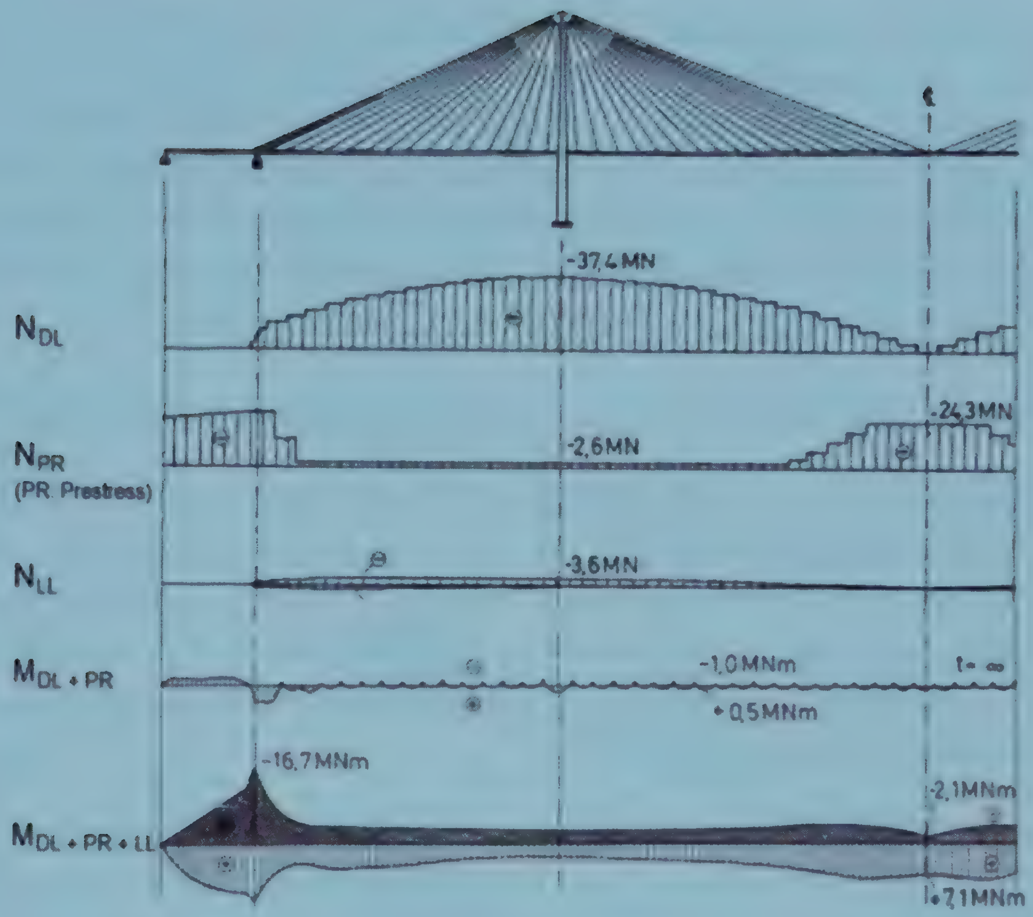


Fig. 2.9 Bending Moments and Compression Forces in Beams of Cable Stayed Bridges [1]

2.3 Advantages of CSB

- The major advantage of CSB is that the stay cables which are a part of the completed structure also serve the function during the construction stages. The flow of forces in the stay cables remains by and large the same.

- Another main advantage of CSB is that the bending moments are greatly reduced by the load transfer of the stay cables similar to a beam on elastic foundation, **Fig. 2.8** By installing the stay cables with their predetermined precise lengths, the support conditions for a beam elastically supported (idealised as non-yielding support) at the cable anchor points can be achieved and thus the moments from permanent loads are minimized, **Fig. 2.9.**

- Large compression forces in the beam are caused by the horizontal components of the inclined stay cables. The normal forces in the main and side span equal one another so that only uplift forces have to be anchored in the abutments which act as hold-down piers.

- Negative live load moments may occur over the vertical bearings at the towers. These can be avoided by supporting the beam by the stay cables only, even in the tower region. The biggest positive and negative moments occur in the side spans near the hold-down piers, which may require special measures. Shear forces remain small.

- The largest horizontal components belong to the backstay cables, as seen in **Fig. 2.9,** the beam compression forces thus increase quickly at the beam ends and increase less quickly near the towers where they reach a maximum. For concrete decks this compression force is beneficial. Only in the center of a 3-span configuration, where the compression forces from cables are small, longitudinal post-tensioning may be required, which also has to overcome tension from friction, braking, etc.

2.4 Comparison of CSB with Suspension Bridge and Extradosed Bridge

A Suspension bridge has a system where there are two sets of main suspension cables, acting as a catenary, connected to pylons on either side of the span. Pylon heights are shorter than for cable stayed bridges, as the pylon of a suspension bridge has to mainly cater to sag of the catenary. This catenary suspends many auxiliary vertical cables to suspend the deck at close intervals. This type of system is suitable for larger spans of say 500 m to 2000 m.

Fig. 2.10 draws a comparison between steel deck cable-stayed and suspension bridge quantities of similar span lengths for a preliminary estimation based on a theoretical exercise. At 1000 m main span, the quantities of steel suspension bridge are more than twice that in a CSB, which demonstrates the efficiency of CSB.

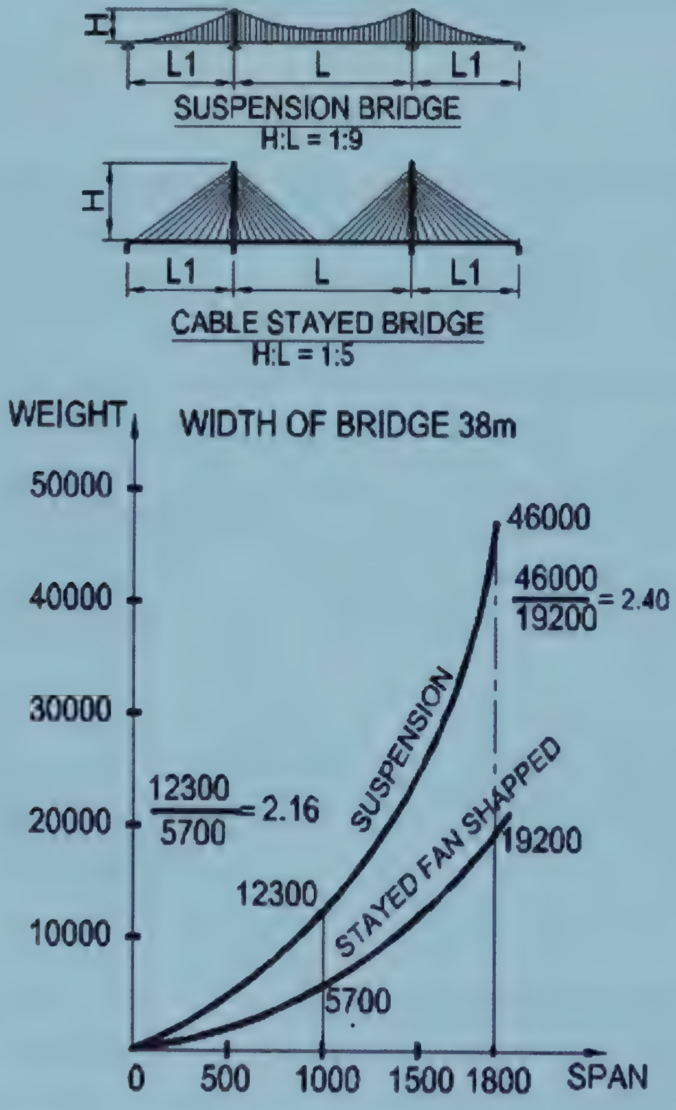
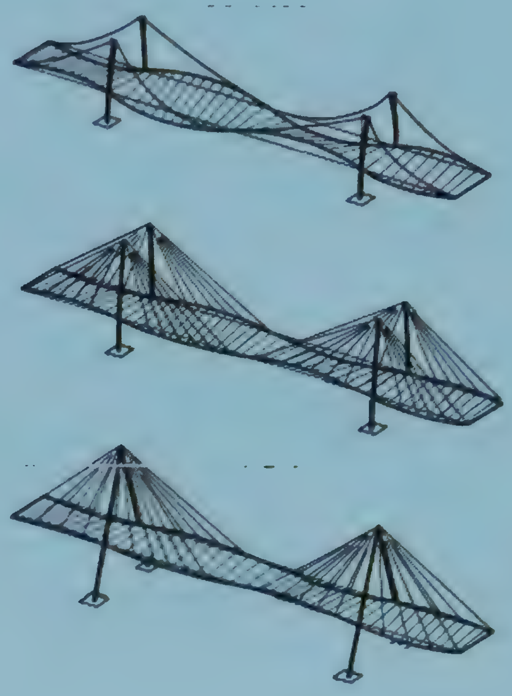


Fig. 2.10 Quantity vs Span Length for Steel / Steel Composite Cable Stayed and Suspension Bridges [5]

Although, in theory the design of span length 1400 m or more is possible with cable stay bridges, but in practice, for span ranges 1200 m-1400 m the choice between suspension and cable-stay will depend upon a number of factors such as topography, construction aspects, economy, etc. Recently, hybrid of suspension – cable stay bridge system has been revived for very long spans, for example Sultan Suleiman bridge (1500 m main span) in Turkey, Messina Straits Crossing (3300 m main span) in Italy, which are on the anvil.



a) Suspension bridge

b) Cable stayed bridge with twin towers

c) Cable stayed bridge with A- tower

Fig. 2.11 Eigen modes of Suspension Bridges and Cable-Stayed Bridges [5]

Cable-stayed bridges are inherently stiffer than suspension bridges. This is especially true for longitudinally eccentric loads, for which the main cables of a suspension bridge find a new equilibrium without increase of cable stresses (stressless deformations), whereas stay cables always have to be additionally stressed in order to carry any load.

Suspension bridges are thus generally not suitable for high railway loads due to which deformations become too large.

The eigen frequencies of cable-stayed bridges are significantly higher than those of suspension bridges. The torsional frequencies, which are especially important for the aerodynamic safety against flutter, can further be increased by using A-towers which omit the elasticity of the backstay cables, **Fig. 2.11**.

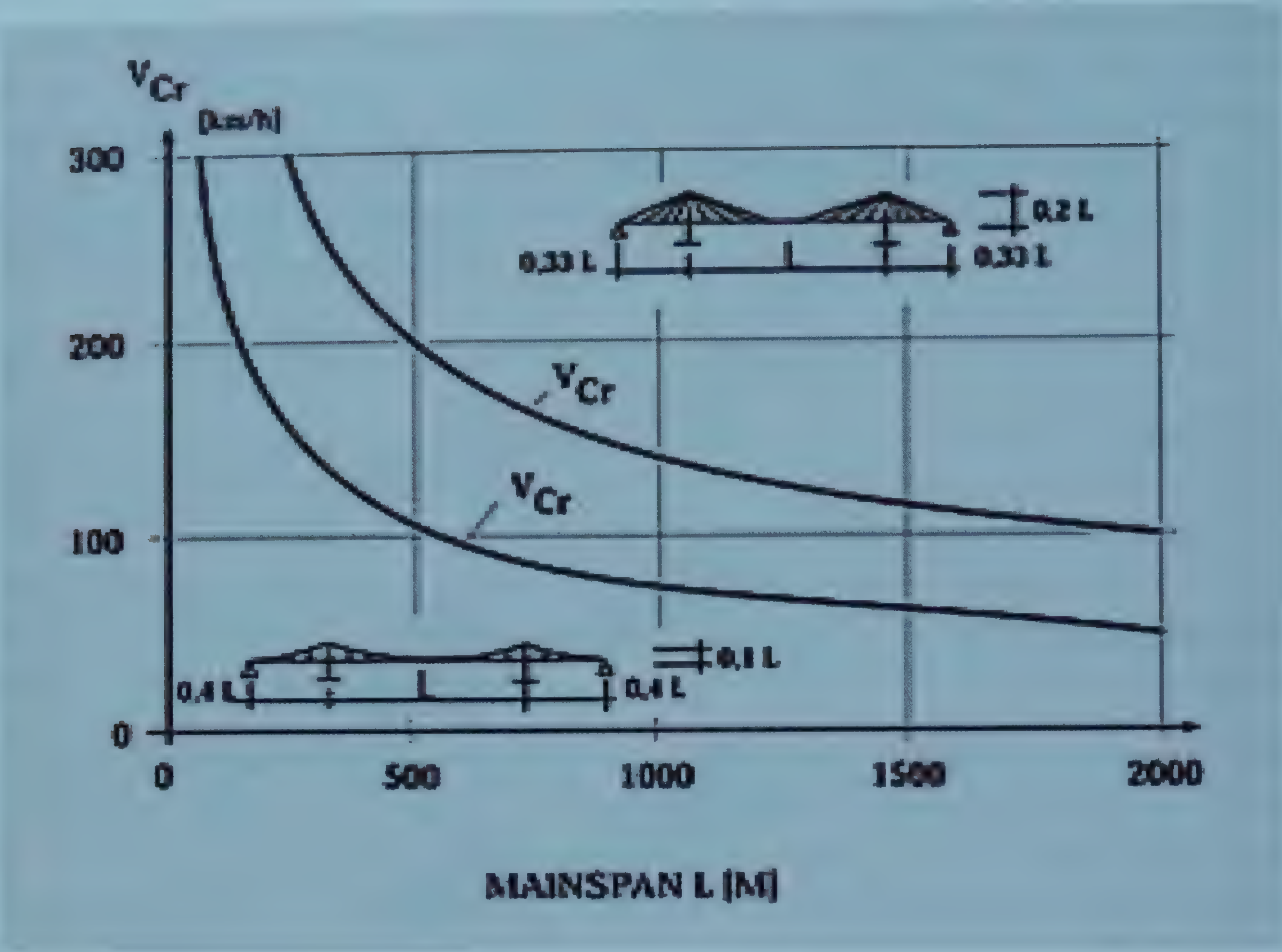


Fig. 2.12 Critical wind Speeds of Suspension Steel / Steel Composite Bridges and Cable-Stayed Bridges [1]

The critical wind speed for the onset of flutter instability due to wind induced vibrations is thus higher for cable-stayed bridges than for suspension bridges. The diagram in **Fig. 2.12** shows that for a main span of 500 m the critical flutter wind speed for a typical cable-stayed bridge is about 200 km/h while for a corresponding suspension bridge it is only 100 km/h.

Cable stayed bridges are quite different from extradosed bridges. Basically, extradosed bridges are in between girder bridges and cable stayed bridges. The extradosed cables are in fact externally prestressed cables. The cables and anchorages used for extradosed bridges are usually different compared to cable stayed bridges, for the reasons of economy.

Extradosed bridges are suited to a smaller span range, say 50 m to 250 m. Similar to cable stayed bridges, these require end spans of about half of central main span. EDs are more rigid and less susceptible to cable vibrations and cable fatigue compared to CSBs. Required height of pylon is the least among the three forms of cable supported bridges. Extradosed bridges are comparatively economical for span ranges of 100 m to 200 m.

Annexure A1 of this guideline gives a table containing broad details of most of the cable stayed bridges in India so far which gives an idea that India already has cable stayed bridges for 40 years.

CHAPTER - 3

STRUCTURAL FORM

3.1 Structural Scheme & Span Configuration

The economic main span range for Cable-Stayed Bridges lies between 200 m and 1100 m. The arrangement of stay cables is influenced by the relationship between the main span and the side span and the location of backstay anchorage. Usually, a module for CSB will comprise of 3 span continuous structure with two towers. However, there are exceptions and cable-stayed bridges can also have two spans with single tower as well as multiple spans.

For conventional 3-span modules, the ratio between the side span and main span has an important influence on the stress changes in the backstay cables which hold the tower back to the anchor pier. Live load on the main span increases these stresses whereas live load on the side span decreases them. These stress changes must be kept safely below the fatigue strength of the cables. From consideration of fatigue strength of stay cables and live load/dead load ratio, the side span is generally kept about 40% in length of the Main span.

Modern cable-stayed bridges have used both steel and concrete material in deck and pylons **[1]**. Configuration for towers include single- and double-plane vertical, A-shaped, and double-plane sloping towers. Deck schemes include cast-in-place concrete, precast concrete, orthotropic steel decks, composite decks, and prefabricated steel. Cable configurations include radiating, harp, and fan. Cable systems use high-tensile bars, parallel wires, or steel bars protected by polyethylene or steel pipe.

Finalisation of the structural scheme and span configuration of a cable stayed bridge is carried out at the planning and conceptual design stage of the project. The concept design evolved must meet the owner's established requirements of aesthetics, operability, economics, maintainability, and functionality. Bridge design requires involvement of experienced personnel knowledgeable in state-of-the-art techniques and methodologies. Important considerations for a cable-stayed bridge design include:

a) A concept which is safe and technically sound,

b) A construction and erection method, which is safe, practical, construction friendly.

c) A structure with low maintenance needs and with a long service life,

d) A structure with aesthetically pleasing form, with low impact on the environment, and

e) A design that permits a faster construction time compared to an alternative comprising the same material and manpower input.

Cable-stayed bridges may be classified by the type of material they are constructed of, by the number of spans stay-supported, by transverse arrangement of cable-stay planes, and by the longitudinal stay geometry.

There are two main categories of modern concrete cable-stayed bridges, namely Rigid Superstructure and Flexible Superstructure, which is explained in detail in **Chapter 2** and shown in **Fig. 3.1** below.

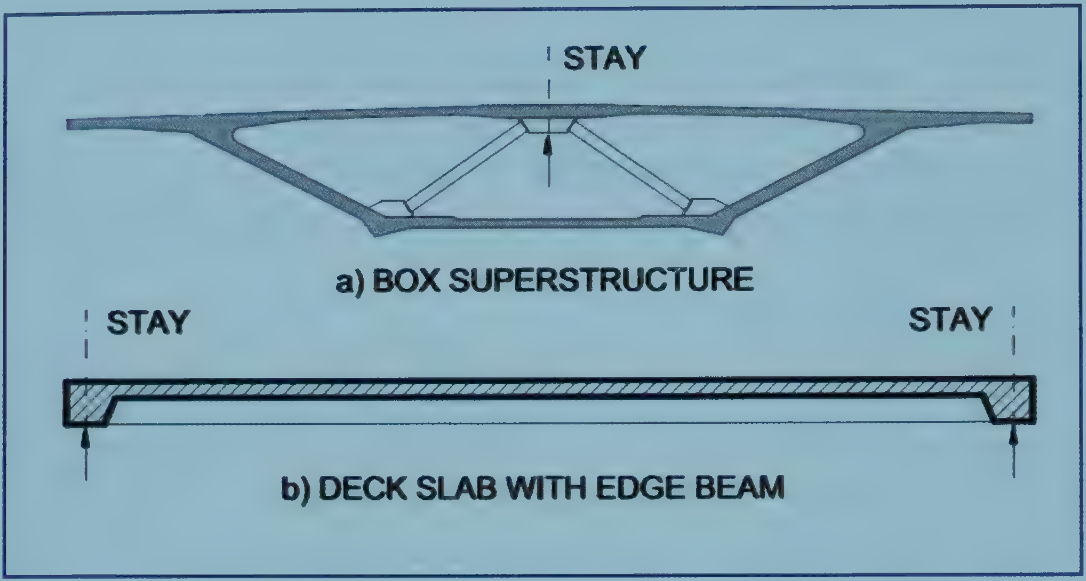

STAY
a) BOX SUPERSTRUCTURE
STAY
STAY
b) DECK SLAB WITH EDGE BEAM

Fig. 3.1 Categories of Modern Cable Stayed Bridges

3.2 Tower Shapes

- A-shaped Pylon
- Diamond Shaped Pylon
- Needle Pylon
- Goal Post Pylon
- H-Shaped Pylon

Fig. 3.2 shows various shapes

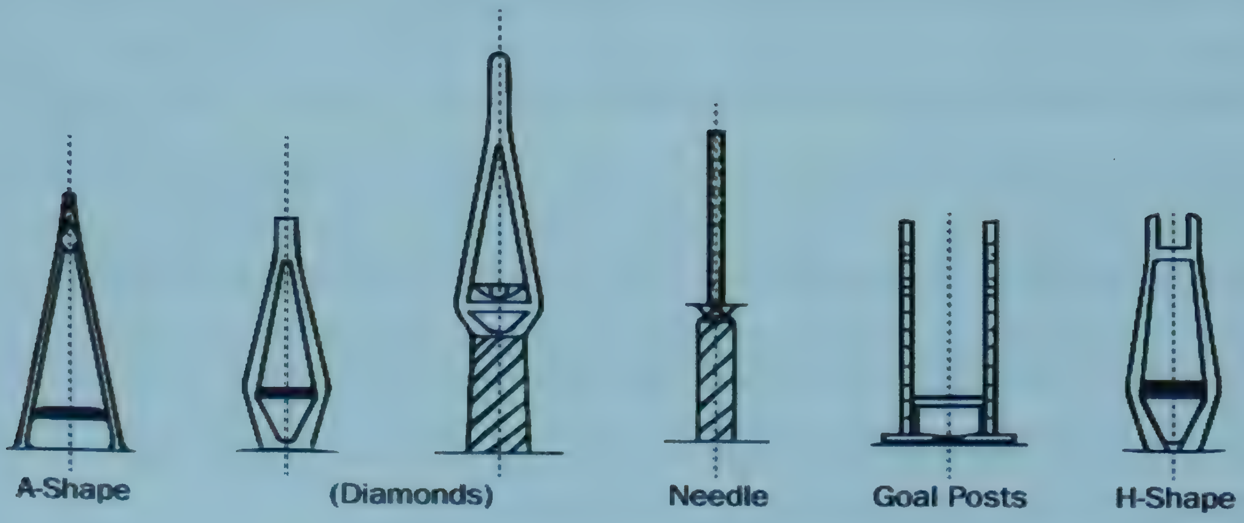

A-Shape
(Diamonds)
Needle
Goal Posts
H-Shape

Fig. 3.2 Categories of Modern Cable Stayed Bridges

Towers constructed from concrete are normally the most economical system. Steel towers are used only in rare cases where there are special considerations.

In the longitudinal direction, the tower should be slender and have a small bending stiffness to avoid large bending moments and reactions at the foundations. Tower bracing above the road deck should be slender and appear light between the thin cables.

Inclined tower construction requires temporary ties to prevent collapse. The towers may need to be cast with an outward chamber for dead load deflection caused by the inclination.

Towers providing live or dead end cable anchorages should be readily accessible for cable installation, stressing, and grouting. This includes providing sufficient space for personnel and equipment necessary to complete these activities. If tower prestressing exists, the same access criteria is required.

Loads induced on the tower by temporary crane tie-ins need to be checked so that allowable stresses are not exceeded.

3.3 Deck Material

The primary construction materials used in the deck of cable-stayed bridges are reinforced or prestressed concrete, composite concrete-steel, or orthotropic steel decks. The choice of deck material will depend on the overall concept for the structure and its elements: the selection of the appropriate structural system for the bridge considering its specific function, site location, and required spans. A well-selected concept determines the efficiency and economy of the bridge, saves materials, cost, and construction time. Good design concepts minimize problems and future difficulties both in the design office and on the construction site.

3.4 Deck Cross Sections

Deck cross-sections are classified by the type of material used in the construction: concrete, steel, and composite steel-concrete. In addition, deck cross-sections may also be classified as a hybrid deck when the main and side spans have different materials. In this case, the main span is usually made of steel in order to be lighter in weight, and the side spans are in concrete and work as a counterweight to provide stability for the main span. **Fig. 3.3** to **Fig. 3.5** show some deck cross-section geometries that are used depending on the type of material.

Besides the geometry, the length of the cable-stayed bridge main span has a direct relation to the type of material. Essentially, concrete decks are for short spans (upto say 400 m), steel decks are for long spans (for spans beyond 700 m) and composite steel-concrete are used for intermediate spans. The record of longest main span of cable-stayed bridges **Table 3.1** is 530 m, 616 m and 1104 m for concrete, composite and steel deck cross-sections, respectively.

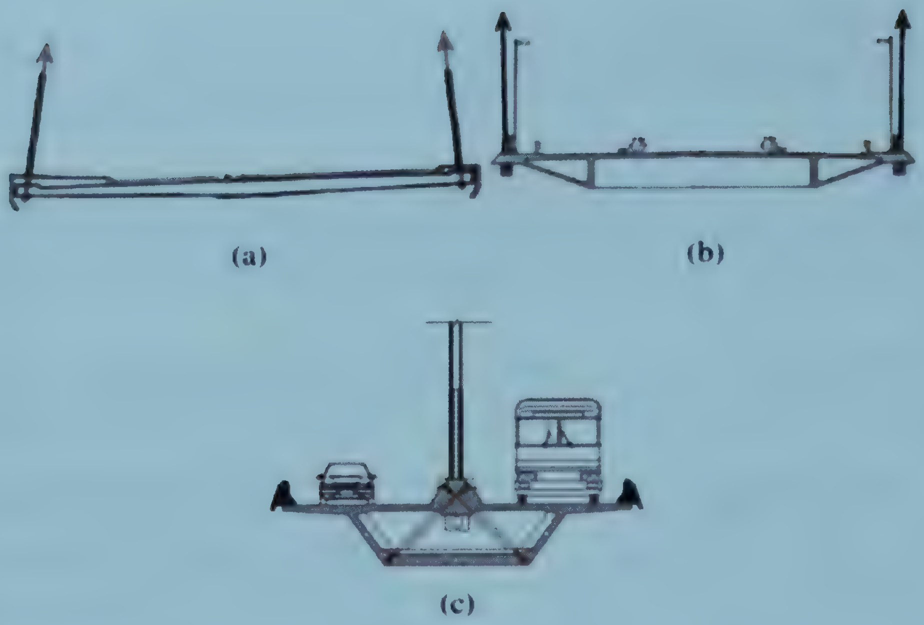


Fig. 3.3 Concrete cross-sections: (a) Thin Concrete Beams; (b) 2 Concrete Girders (c) Box Girder [1]

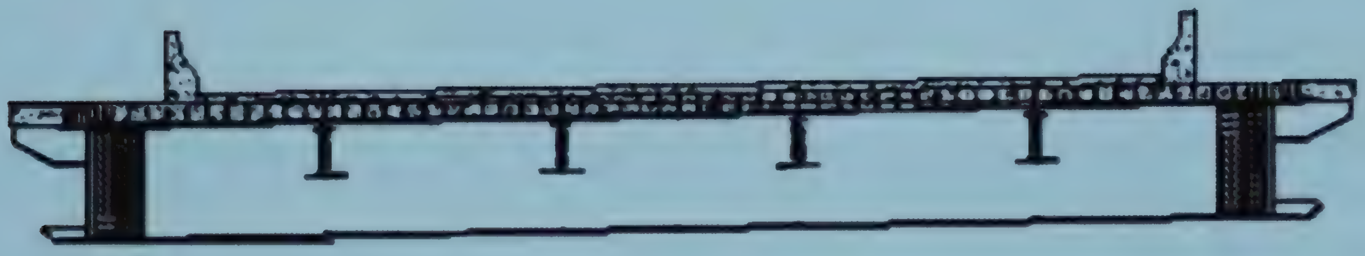


Fig. 3.4 Composite Steel-concrete Cross-Section: Two Main Plate Girders

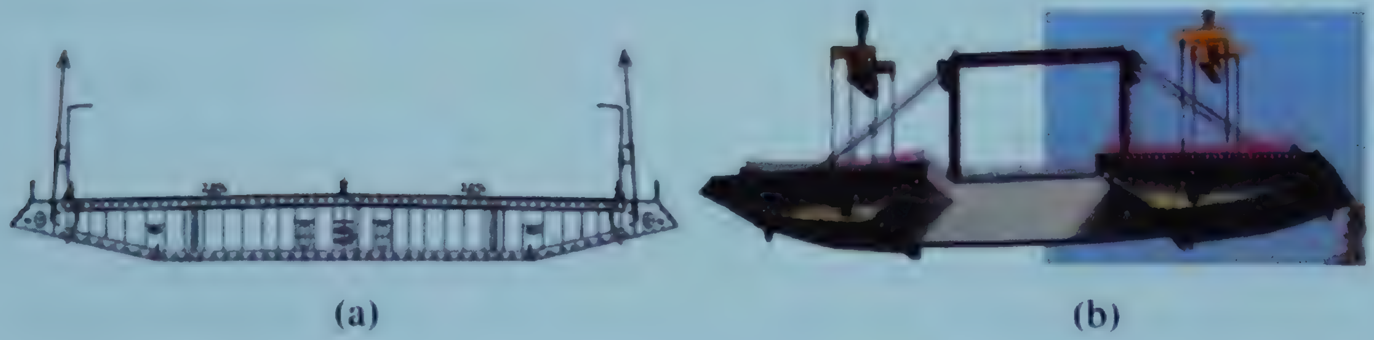


Fig. 3.5 Steel cross-section: Box Girder [1]

Table 3.1 Record Main Spans for Cable-Stayed Bridges

Material	Bridge	Country	Year	Main Span (m)
Concrete	Skarnsundet Bridge	Norway	1991	530
Composite	Erqi Yangtze River Bridge	China	2011	616
Steel	Russky Island Bridge	Russia	2012	1104

Table 3.2 shows the relation between type of cross-section and main span size. To develop this relation, it was assumed that short and medium main span lengths are under 200 m and 450 m, respectively, while long spans are over 450 m.

Table 3.2 Relation Between Main Span Length and Cross-Section Material and Geometry

Material	Geometry	Main Span Length		
		Short	Medium	Long
Concrete	Thin Concrete Beams	√		
	2 Concrete Girders	√	√	
	Box Girder		√	
Composite	2 Plate Girders	√	√	√
	3 Plate Girders		√	
	4 Plate Girders		√	
	Truss Deck		√	
	2 Box Girders			√
	Hybrid			√
Steel	Box Girder		√	√
	2 Box Girders		√	√

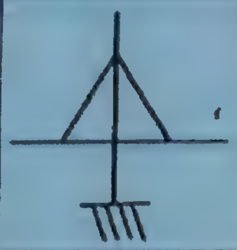
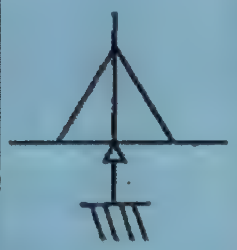
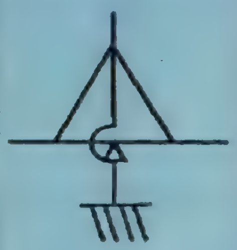
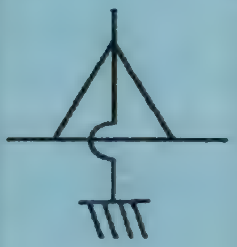
In selecting the cross section of the deck for cable stayed bridges, in addition to the bridge type and load, the number of stay cable planes, the way they will be anchored, and the transfer of tension between the cables and the girder shall all be considered. Also, the deck cross section shall be determined with consideration for wind resistant stability.

3.5 Connection between Pylon, Pier and Deck

- Pylon, Pier and Deck are all rigidly connected
- Pylon and Pier are rigidly linked. Deck is free or support bearing is put under the deck
- Pylon and girder are rigidly linked, and support bearing is put on pier top.

Table 3.3 gives the comparison between various types of girder support and the connection between Pylon, Deck and Pier **[2]**.

Table 3.3 Comparison of Girder Support Types and Connection Methods for Pylon, Pier & Girder

Type of girder support	Connection of pylon, pier and girder	Outline Diagram	Characteristics
Rigid frame type	Pylon, pier, and girder are all rigidly connected		• Bearing is unnecessary, suitable for cantilever construction method. • If more than 3 spans, it takes large effects by temperature change, creep, and shrinkage. • Little displacement during earthquakes but sectional force of the girder in the region near the pier top is large.
Continuous girder type	Pylon and girder are rigidly connected; Pylon and girder are supported by bearing		• Reaction force distribution for piers becomes easier. • Cross section of girder and pylon in the region of connection section becomes larger • The bearing which supports the pylon and girder need a large scale. • During cantilever construction temporary fixing is necessary. • Sectional force of superstructure is small during earthquakes. • The first mode is dominant during earthquakes and natural period becomes shorter. • Acting position to the substructure becomes low
	Pylon and pier are rigidly connected; girder is supported by bearing		• It is possible to distribute the reaction force to piers • The bearing which supports the part of girder can be done by small scale. • Suitable for continuous type girder structures • During cantilever construction temporary fixing is necessary. • Sectional force for pylon and pier becomes large during earthquakes.
Floating type	Pylon and pier are rigidly connected; girder is not supported by bearing		• Long natural period, can lighten earthquake forces but large girder displacement. • Bearing support is not necessary but the bearing fixing the girder in direction perpendicular to bridge axis is separately necessary. • During cantilever construction temporary fixing is necessary. • Limited to multi-cable cable-stayed bridges. • Sectional force for pylon and pier becomes large during earthquakes.

3.6 Stay Cable System and Technology

- Number of Cable Plane
  - Single Plane
  - Double Plane
- Arrangement of Cables
  - Harp Shape
  - Fan Shape
  - Radial Shape (Parabolic Shape)

There are three basic cable arrangements in bridge elevation. With the fan arrangement, all cables join at the tower head **Fig. 3.6.** With the harp arrangement, all cables run parallel and are anchored over the height of the tower **Fig. 3.7.**

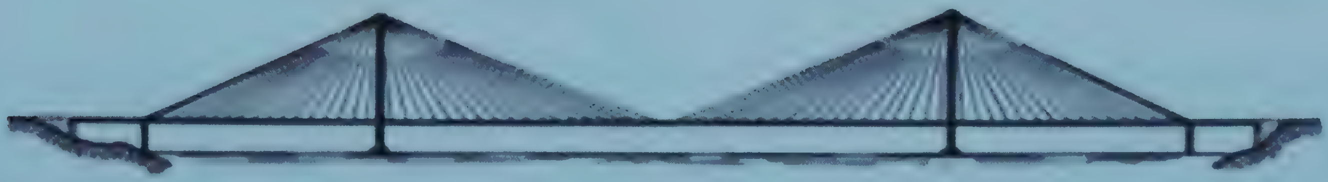


Fig. 3.6

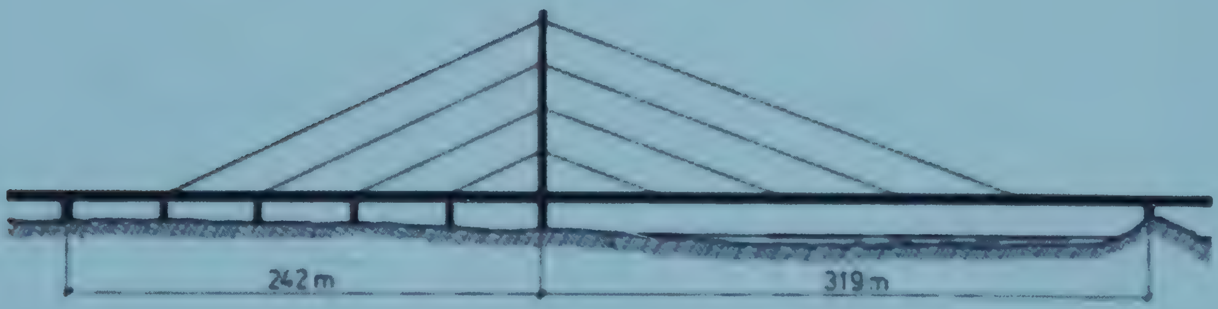


Fig. 3.7

The true fan arrangement cannot be realized practically since in the case of damage, each individual cable must be able to be exchanged by reversing the installation procedure, which is difficult in true fan arrangement. This exchangeability requires individual cable anchorages to have certain minimum spacing between them at the tower head.

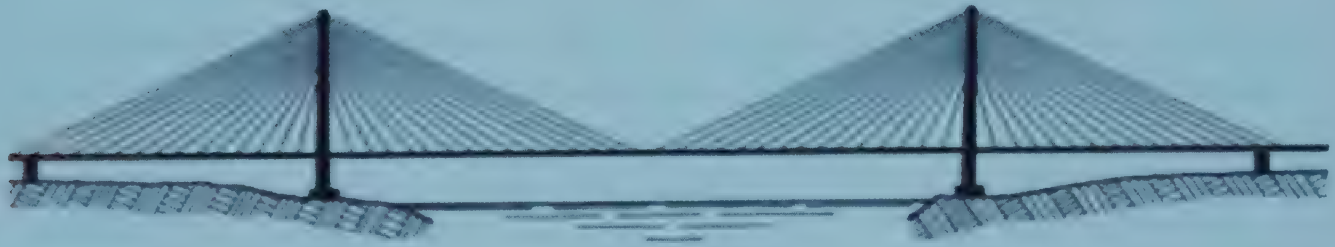


Fig. 3.8

An intermediate cable arrangement between harp and fan is created, as shown in **Fig. 3.8**, which should be as close as possible to the fan arrangement due to its economical advantages. The harp arrangement, however, is aesthetically advantageous because there are no visual intersections of the stay cables, even for two cable planes in a skew view.

The arrangement of stay cables is also influenced by the relationship between side span and main span, and the location of the backstay anchorages, e. g. by connecting them directly above the piers in the side spans. The harmonic cable arrangement plays an important role in the aesthetic appearance of a cable-stayed bridge and has thus to be studied carefully from the very beginning of a design.

CHAPTER - 4

CONCEPTUAL DESIGN

4.1 The Concept

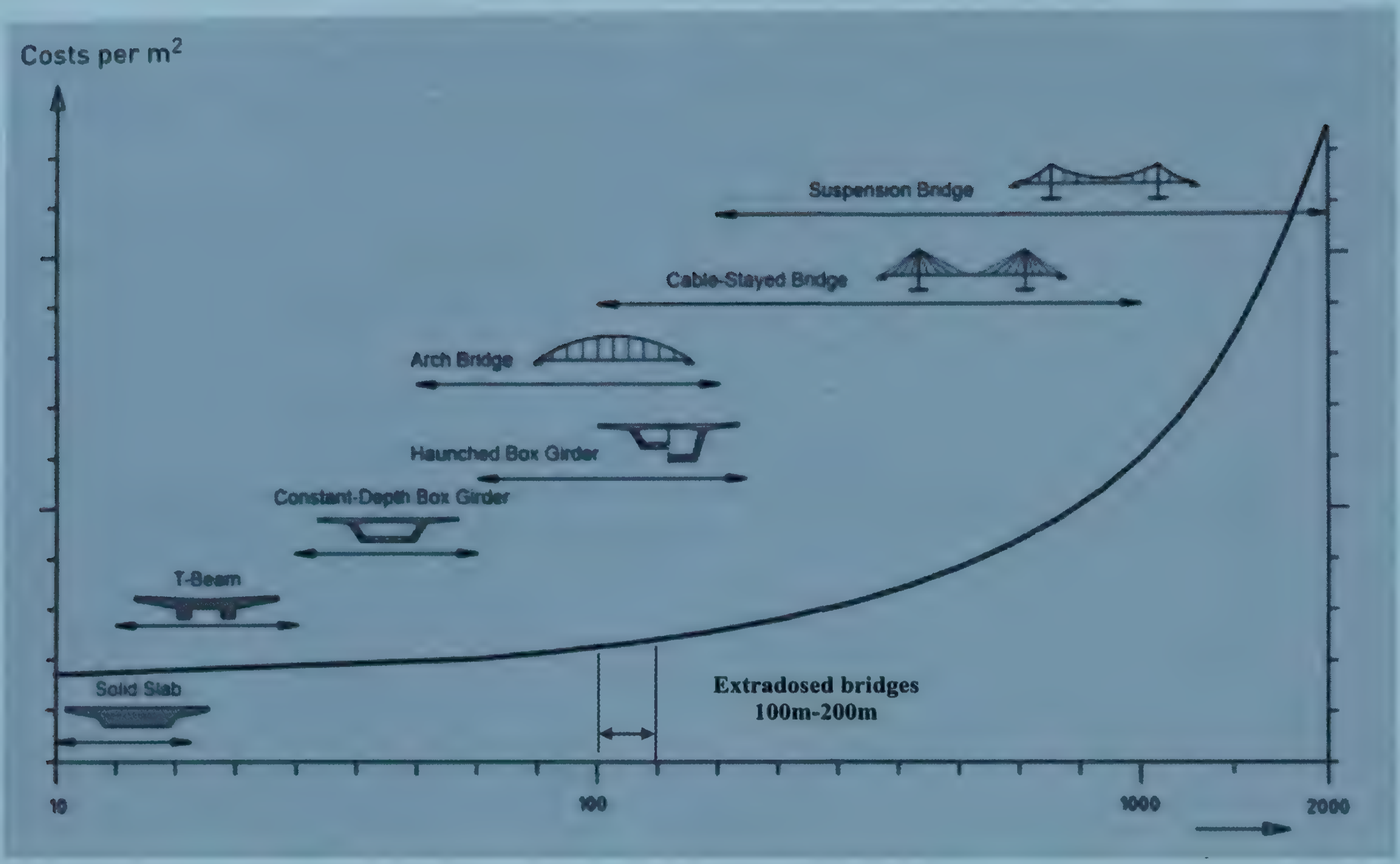


Fig. 4.1 Economical Span Range for Various Bridge Types

Each bridge type has a range of spans in which it is most economical. This is shown in **Fig. 4.1**, modified from Svensson **[1].** Svensson has also come out with the most suitable material for the decks of different span ranges for Cable Stayed Bridges as follows:

Concrete.......	< 400 m
Composite......	400-600 m
Steel.........	> 600 m

Good references for the Preliminary design of Cable Stayed Bridges are already available in several books and papers Svensson **[1]**, Gimsing **[2]**, Walther **[3]**, Troitsky **[4]**, Calvi **[5]**, to name a few. The attempt in this Chapter is to understand the structural concept of Cable Stayed Bridges by viewing it largely through the bridges that have been constructed in India.

The superstructure of Cable stayed Bridges **Fig.4.2** consists of the following:

- Deck
- Pylons
- Stay Cables

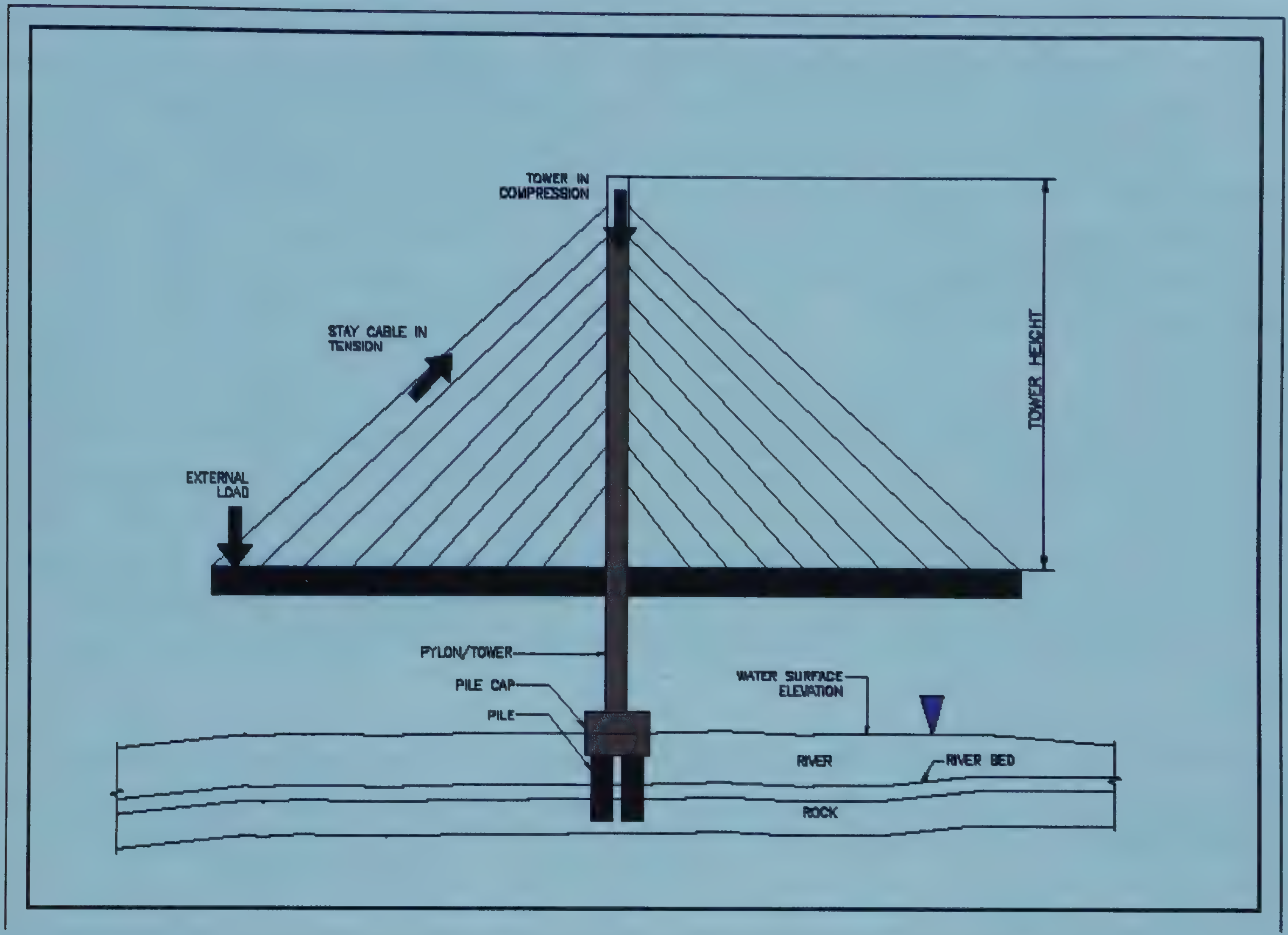


Fig. 4.2 Structural Action of Cable Stayed Bridges

The structural actions of all three are linked when we take account of dead loads of the structure or when a vehicle crosses over the deck. The global structural action can be seen in **Fig. 4.2** clearly. Any load on the deck (including its own dead weight) is carried mostly by the tension in the stay cables and only a small part of it is carried through the flexure of the girder. The vertical loads are transferred to the pylon which is designed to carry the axial compression imposed on it. The Compressive axial forces in the deck caused by the inclined stay cables and the axial compression in the pylons are in equilibrium with the tensile forces in the stay cables. Since the loads are mainly transferred as axial forces rather than bending, it makes the option of Cable Stayed Construction most economical for long span bridges. It is also simpler and safer to construct as the deck can be constructed using the free cantilevering technique without ground support. It is faster to construct because the erection of the deck can be commenced as soon as the pylons reach the height of the first cable anchor point, while the remaining height of the pylon and the deck erection can be carried on simultaneously.

It is highlighted that bearings are almost always preferred to be avoided at tower locations so that hogging moments due to vertical live load do not occur and the deck is ‘cradled’ by the stay cables by providing elastic supports to the deck. Bearings are provided at the end supports i.e. abutments in a three-span arrangement, **Fig. 4.3** for vertical loads apart from providing restraints at all supports for lateral movements transverse to the deck. The deck can move longitudinally under the effect of temperature changes, shrinkage, creep or live loads and hence one of the end bearings has to be made a “rocker” while at the other end a “free” bearing is provided. Given the long length of the bridge, expansion joints capable of large movements become necessary.

The three-span arrangement (**Chapter 3**) is the most common arrangement employed in Cable Stayed Bridges which consists of a main span and two side spans. A comparison of the three span arrangement for the common bridge types **[6]** is shown in **Fig. 4.3**. It may be noted that the side spans of Cable Stayed Bridges used for highways are optimized as being about 40% of the main span in contrast to the Free Cantilever and Extradosed Bridges where they are much longer. The reason is that the sagging effects of the main span in Cable Stayed Bridges often become the governing criteria of design and a shorter side span would provide greater stiffness to the main span. A comparison of midspan effects of the main span for a simply supported beam and a fixed ended beam is shown in **Fig. 4.4** to illustrate the importance of providing greater stiffness at the ends of the main span. As shown, the deflection in the main span in the fixed beam is merely 1/5th of that of the simply supported case, while the bending moments reduce to 1/3rd. However, the short side spans result in a tendency of the deck to uplift at the end supports both due to DL alone as well as LL when applied on the main span. This necessitates the anchoring of the deck at the end support by kentledge or other means which will be discussed later in this chapter. The variation of stress in the backstay cable is another issue which will also be discussed.

In some instances, short multiple spans are provided in the approaches instead of one single side span which improves the situation still further **Figs. 4.5 & 4.6** (Chambal Bridge). There is an additional advantage of this arrangement in that the stay cables can be made symmetrical about the pylon which avoids large stress variation in the backstays. This is discussed later in the Chapter.

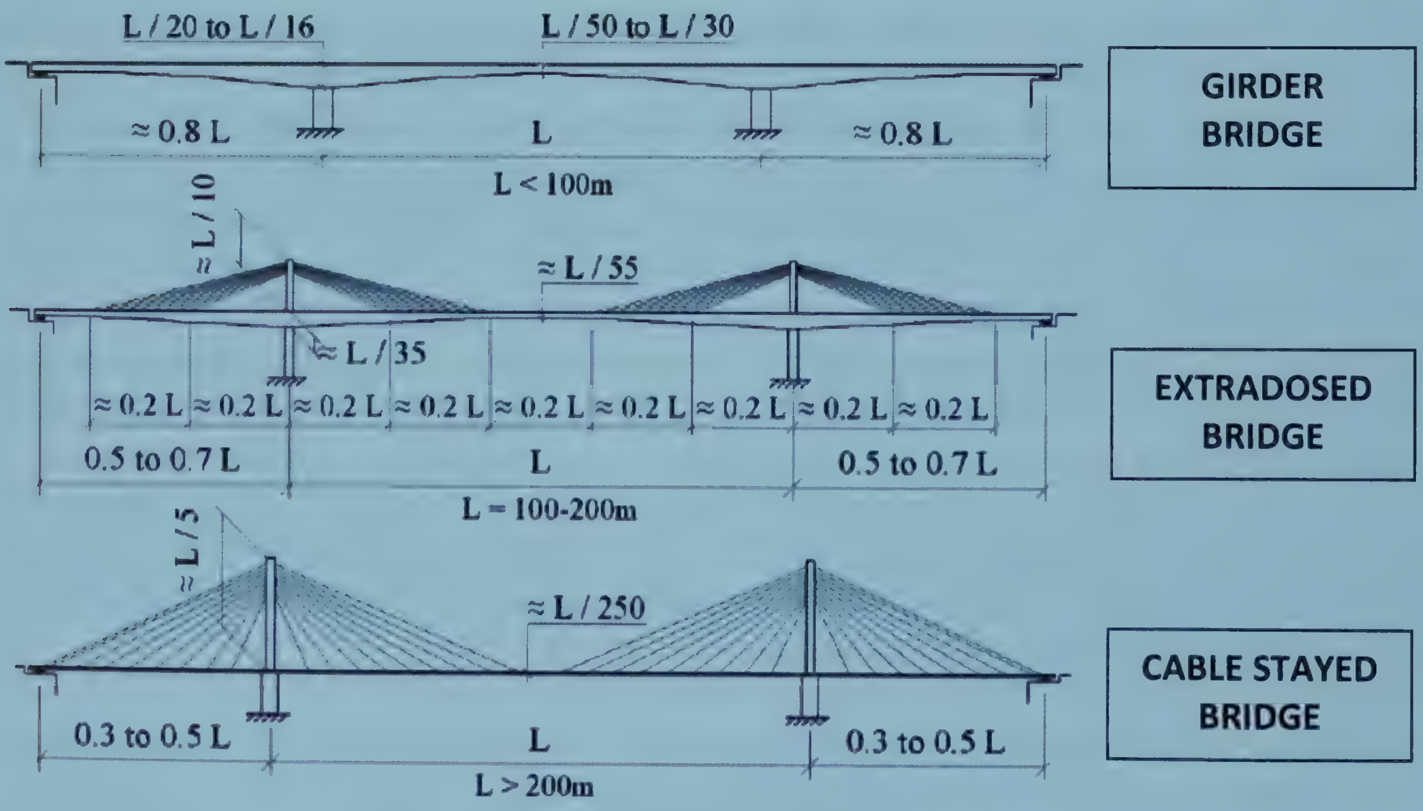
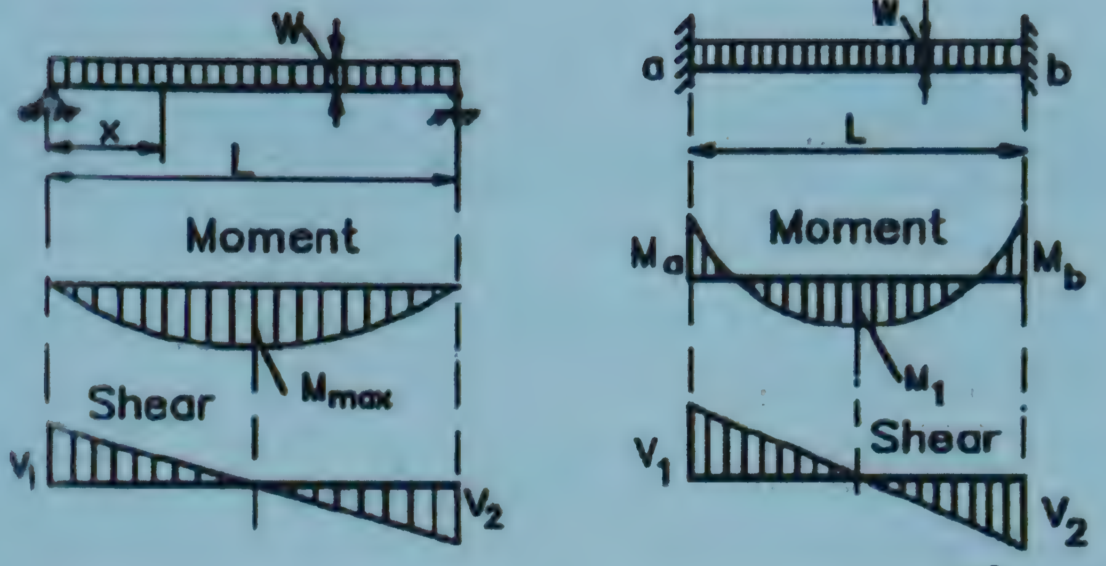


Fig. 4.3 Proportioning of Spans in Different Bridge Types Compared [6]


W
x
L
Moment
Shear
Mmax
V1
V2
W
a
b
L
Moment
Ma
Mb
M1
Shear
V1
V2

$$M_{max} = \frac{wL^2}{8}$$

$$M_1 = \frac{wL^2}{24},$$

$$\Delta_{max} = \frac{wL^4}{384EI}$$

Fig. 4.4. Importance of Stiffness Provided by Side Spans for Reducing Sagging Effects in Main Spans

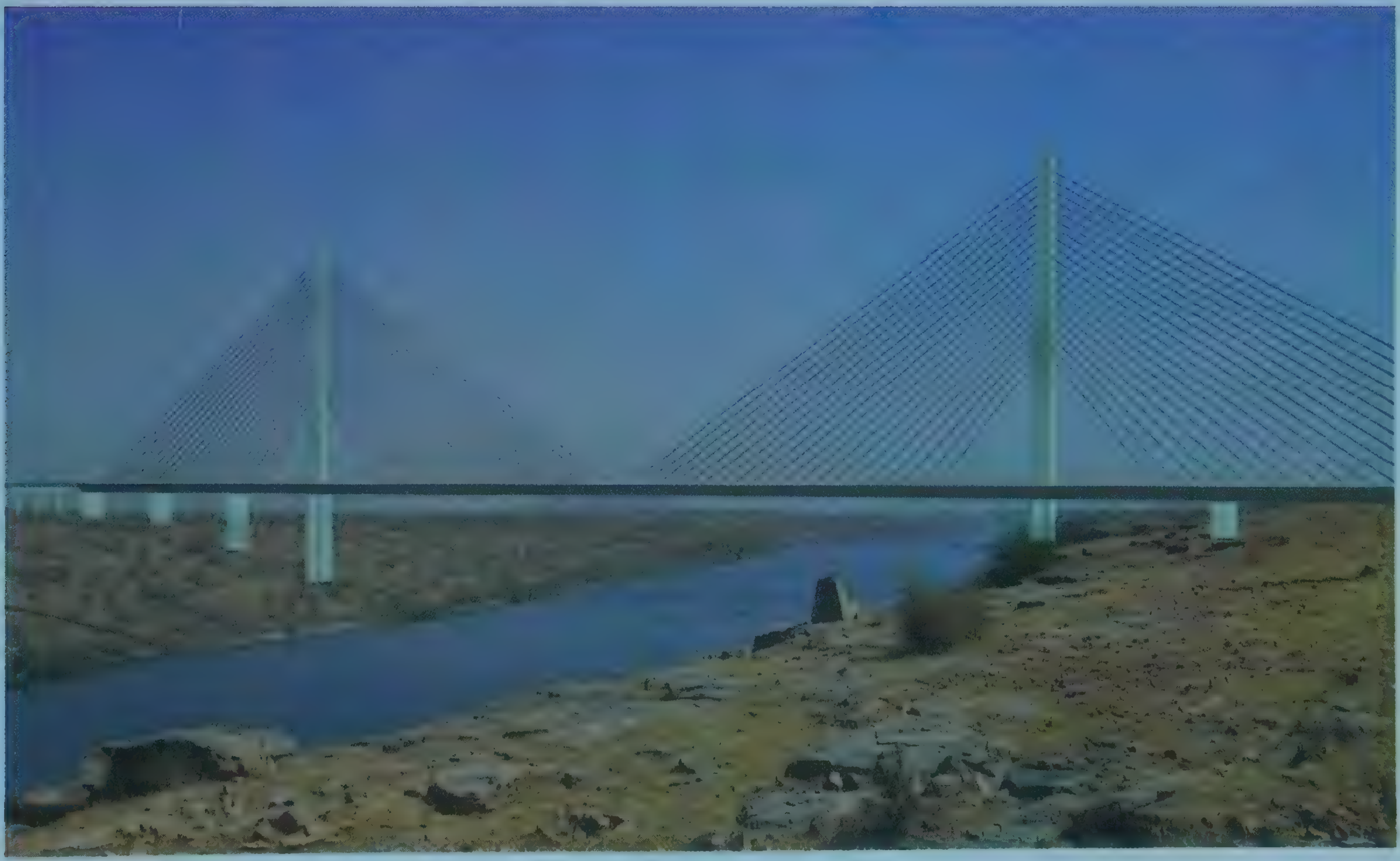


Fig. 4.5: Kota Bridge over Chambal, Rajasthan (350 m)

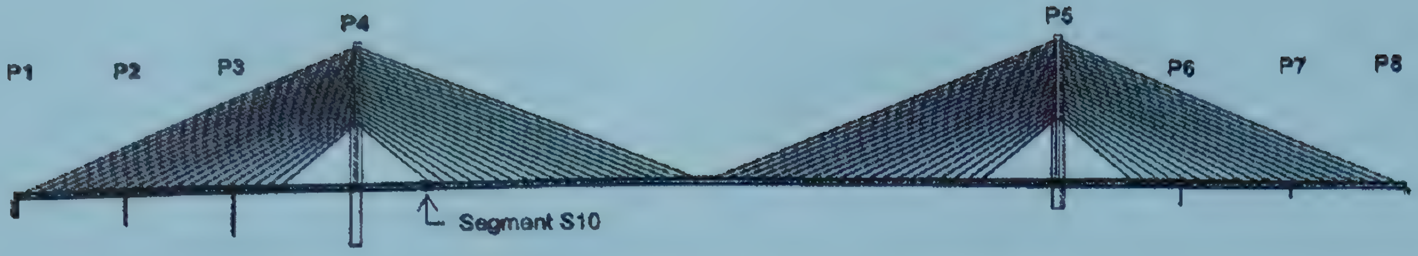

P1
P2
P3
P4
P5
P6
P7
P8
Segment S10

Fig. 4.6. Kota Bridge over Chambal, Rajasthan (Main Span: 350 m, Side Spans 50 - 60 m)

In the case of multi-span cable stayed bridges with equal spans the benefit of short side spans cannot be availed of. The same is the case with Single Pylon bridges supporting two equal spans on either side. Since there are no backstays in such structures which can restrain the tower head, the tower has to be stiff by itself. This is best achieved by towers that are A-shaped in the longitudinal direction, **Fig. 4.7 (a) & (b)** (Worli Bandra Bridge).

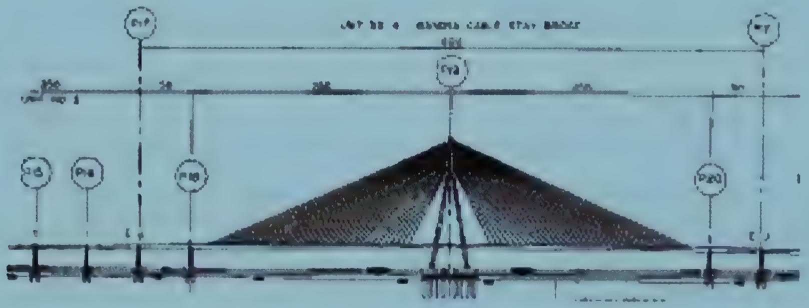
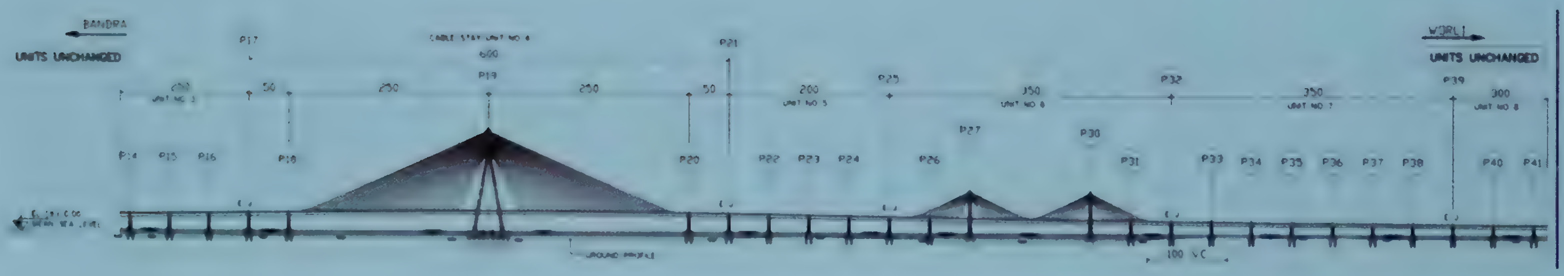
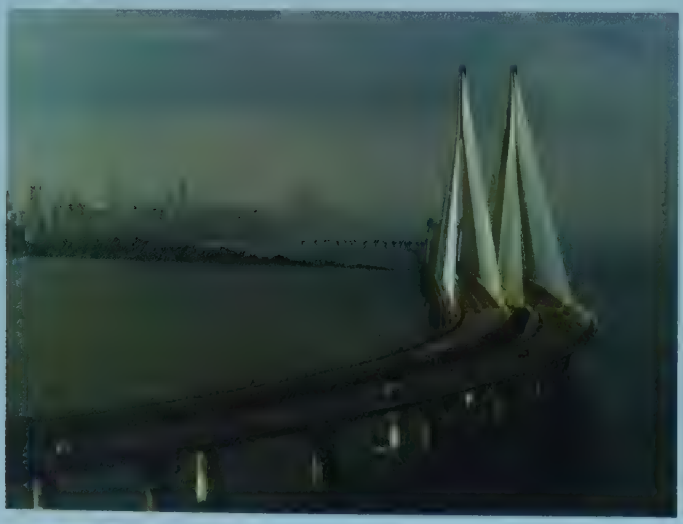

P17
P19
P18
P20
P21

Fig. 4.7 (a) Worli Bandra Sea Link


BANDRA
UNITS UNCHANGED
P17
CABLE STAY UNIT NO 4
600
P21
P19
50
250
250
50
200
UNIT NO 5
P25
350
UNIT NO 6
P32
350
UNIT NO 7
WORLI
UNITS UNCHANGED
P39
300
UNIT NO 8
P14
P15
P16
P18
P20
P22
P23
P24
P26
P27
P30
P31
P33
P34
P35
P36
P37
P38
P40
P41
GROUND PROFILE
100 VC



2*250 m span with Single pylon

Length: 5.6 km
Width : 8 lane traffic

Fig. 4.7 (b) Worli Bandra Sea Link, Mumbai

The design of such bridges is discussed later in this Chapter.

4.2 The deck

The deck is quite thin considering that the span to depth ratio can be of the order of L/250, where L is the main span length. The low depth to span ratio is made possible because of the closely spaced (6 to 12 m) stay cables supporting the deck which provide to it elastic supports at their junction with the deck. The deflections of the deck due to applied loading are kept under control by the presence of the stay cables and are not governed to any significant degree by its stiffness. Thus, a thin deck suffices provided that, it is safe under aerodynamic and seismic effects as well as torsional loading.-The deck can be constructed in situ or constituted from precast concrete segments (**Chapter - 3**). The deck may also be in composite steel-concrete construction **Fig.4.8**, Signature Bridge; **Fig. 4.9 (a)** (Vidyasagar Setu) or structural steel. Free cantilevering technique remains the predominant construction **Fig. 4.9 (b)**.

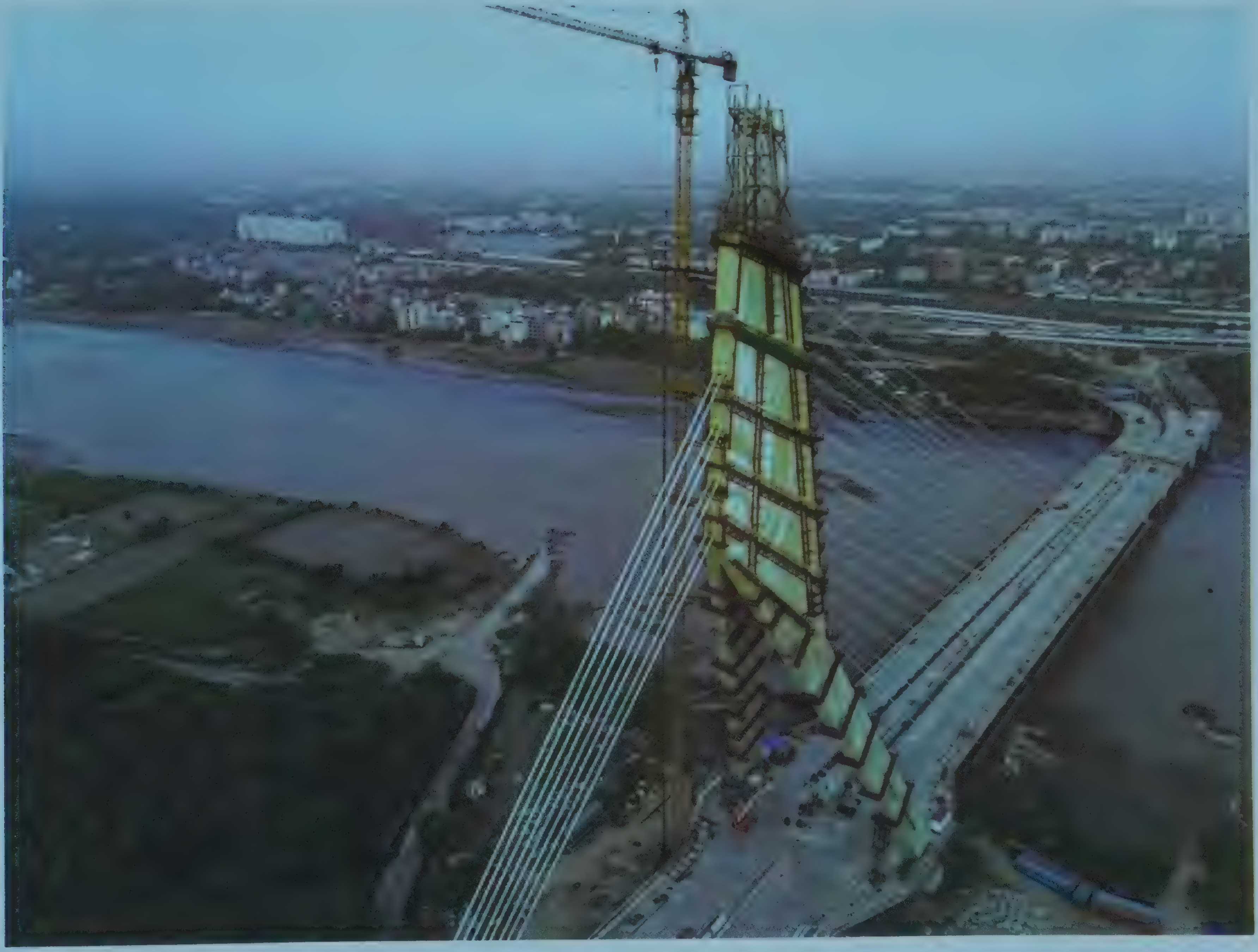


Fig. 4.8: Signature Bridge

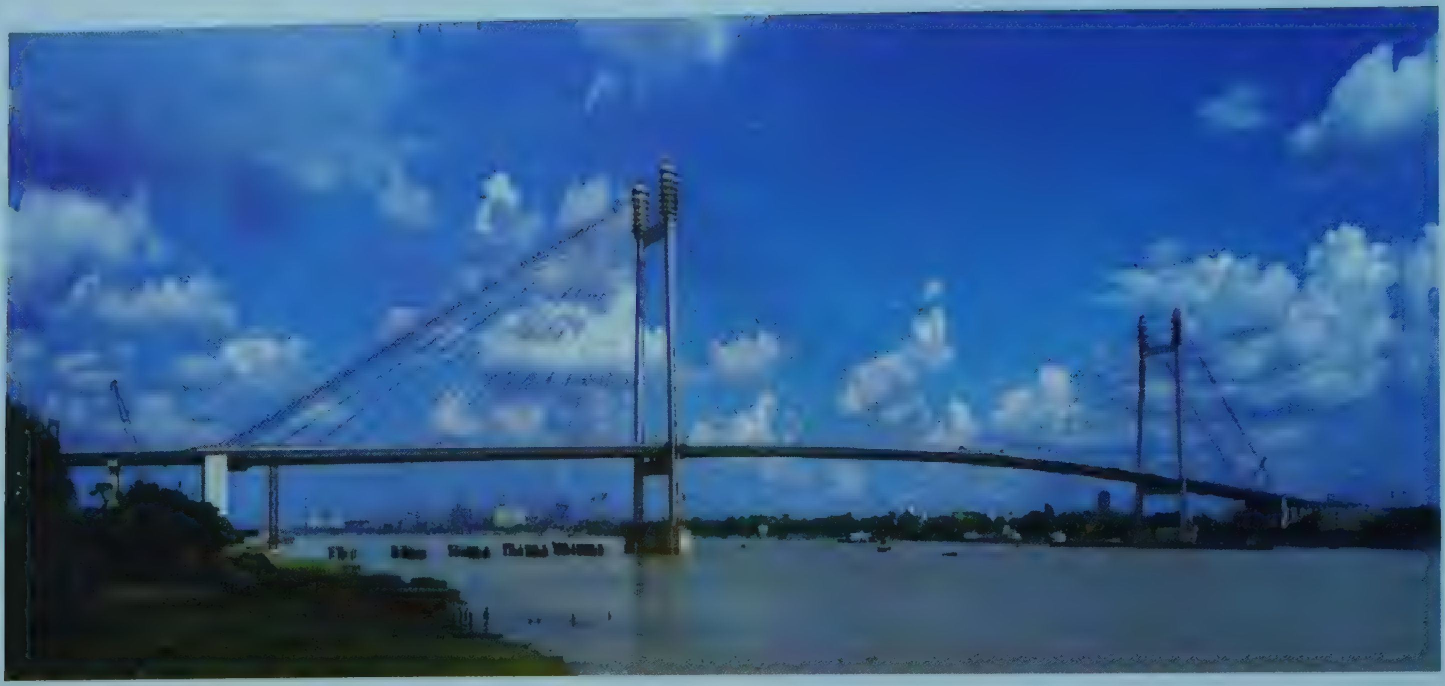


Fig. 4.9 (a) Vidya Sagar Setu over Hooghly, Kolkata (Main Span 457 m)

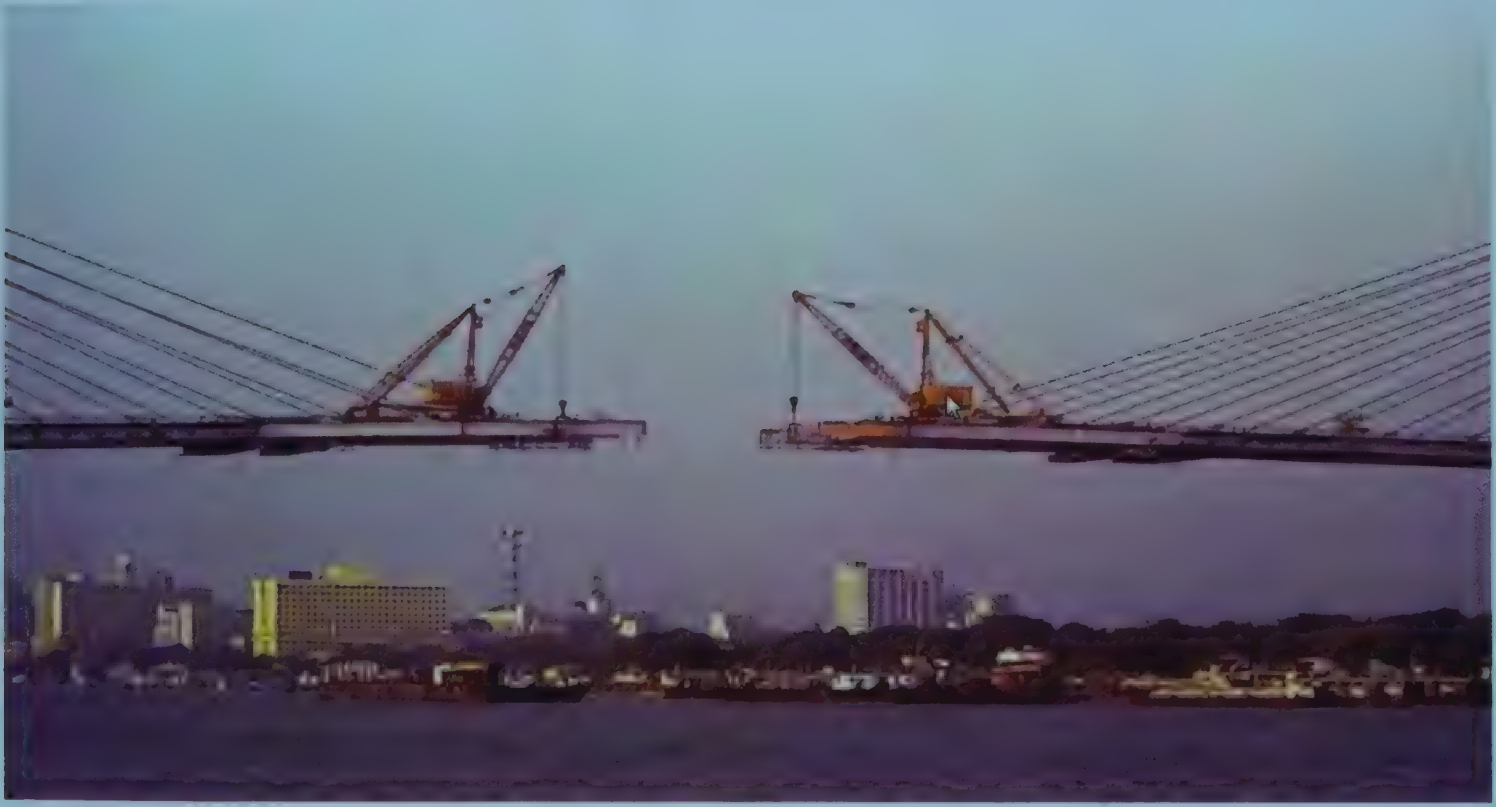


Fig. 4.9 (b) Vidya Sagar Setu over Hooghly, Kolkata (Main Span 457 m). Free cantilevering of the deck

The form of the section of the deck depends largely on the mode of support provided by the stay cables (**Chapter-3**). If the cables support the deck at its two extremities **Fig. 4.9 (a)** Vidyasagar Setu, **4.10 (a), 4.10 (b), 4.10 (c)** Naini Bridge, **Fig. 4.11** Atal Setu, **Fig. 4.12 (a), 4.12 (b)** Signature Bridge by two planes of stays, the section can consist of a simple slab-like structure with or without a grid of beams below it. It is usual to think in terms of a deck without the necessity of providing torsional stiffness as eccentric loads in the transverse sense would be transferred directly to the two planes of stay cables increasing the tension in one plane of stays and reducing tension in the other plane of stays **Fig. 4.10 (b), 4.10 (c)** Naini Bridge.

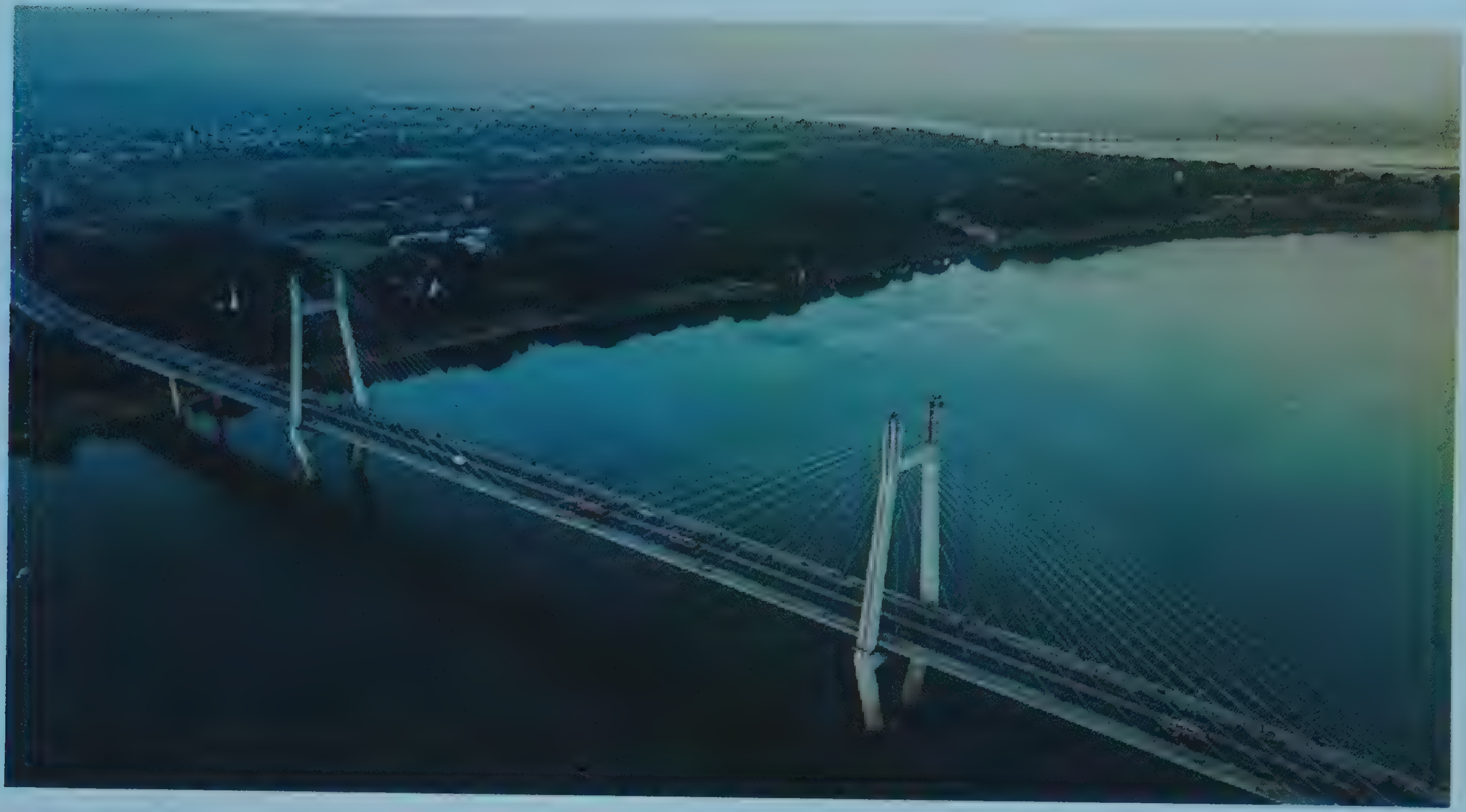


Fig. 4.10 (a) Naini Bridge

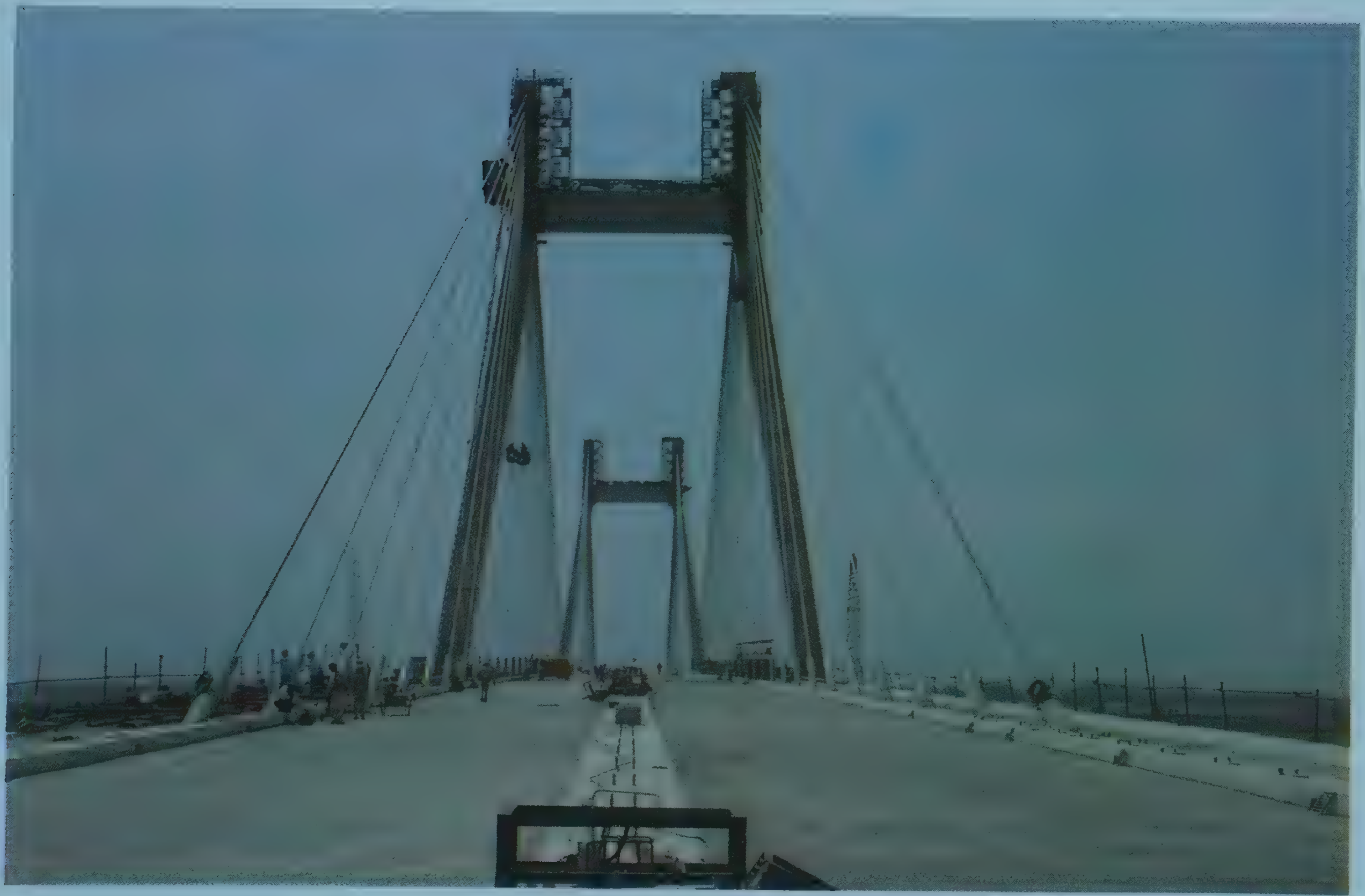


Fig. 4.10 (b) Naini: Naini Bridge Across River Ganga Deck Width 26.4 m, Central Span 260 m

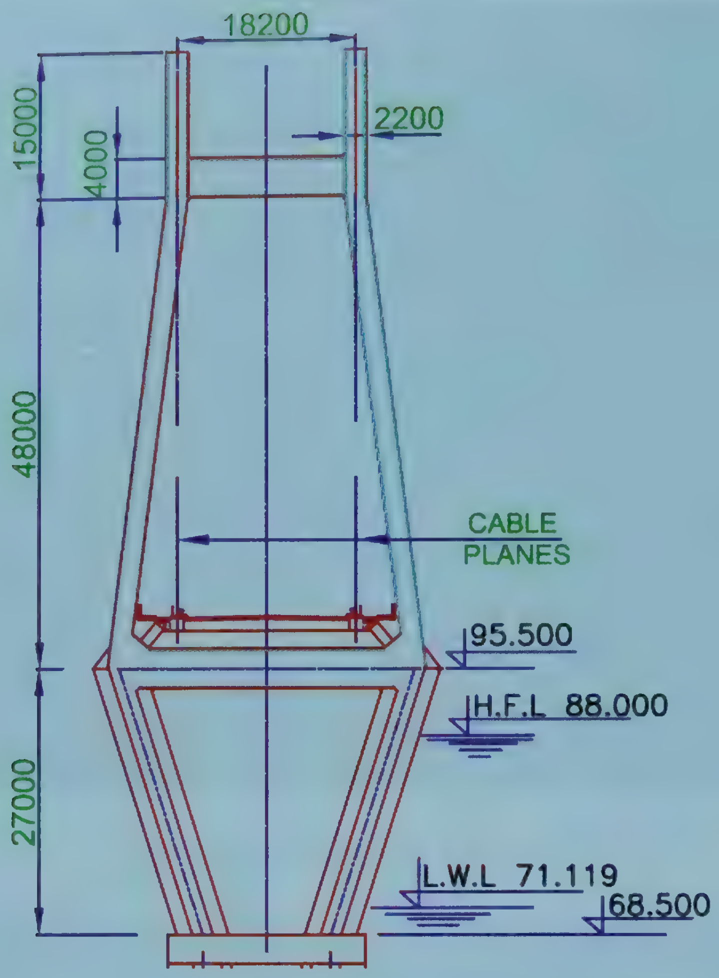

18200
2200
15000
4000
48000
CABLE PLANES
95.500
H.F.L 88.000
27000
L.W.L 71.119
68.500

Fig. 4.10 (c) Naini Bridge

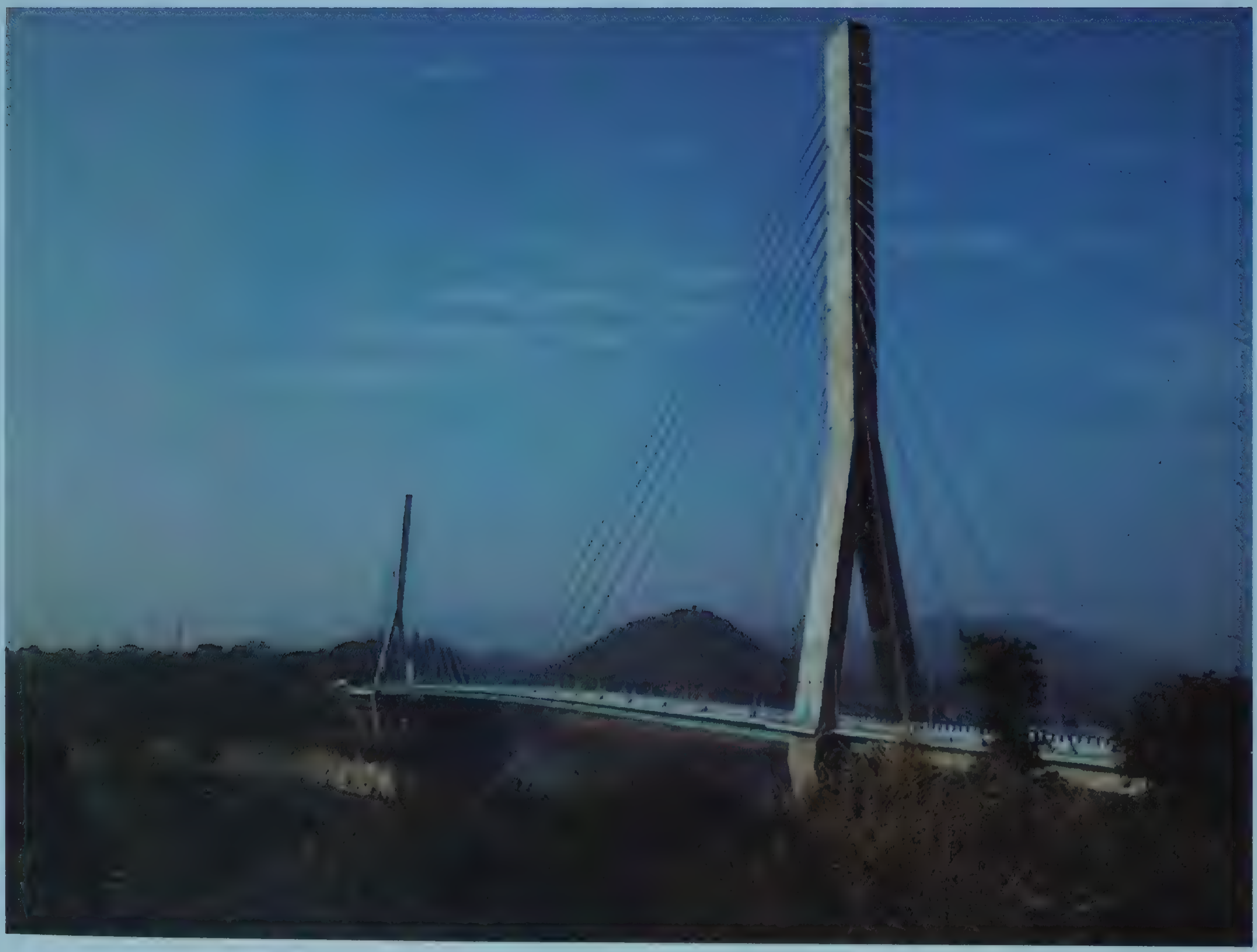


Fig. 4.11. Atal Setu over Ravi, J&K (Main Span 350 m)

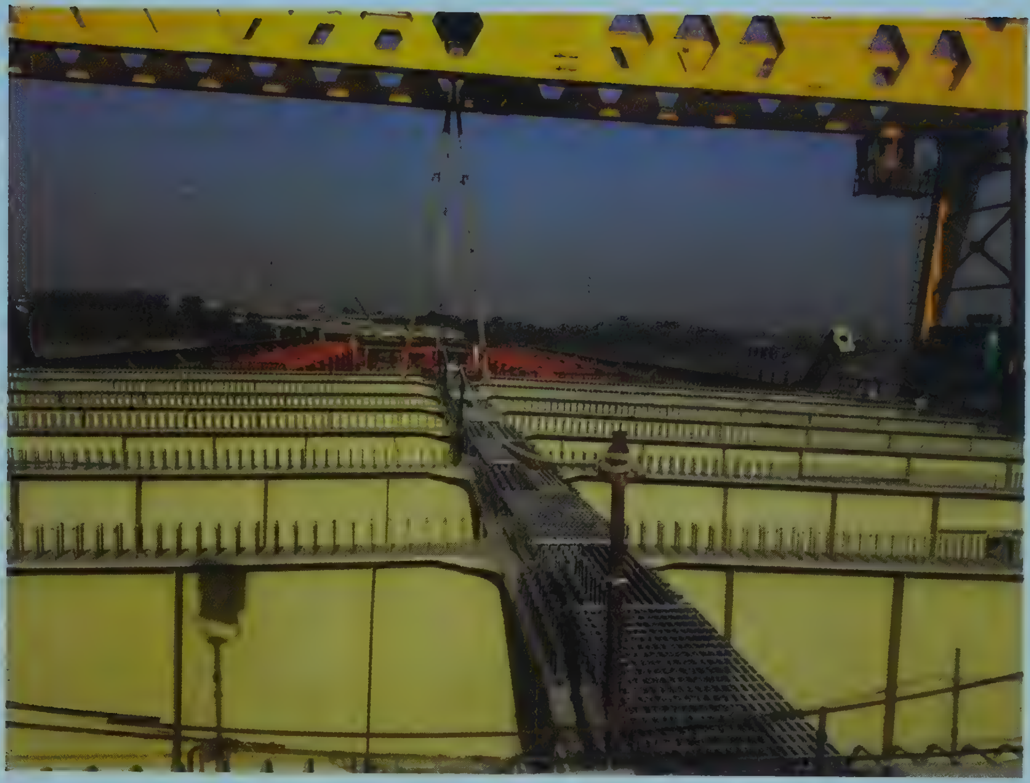


Fig. 4.12 (a) Steel Grillage For Deck (35.2 m wide): Signature Bridge Before Concrete Deck Placement

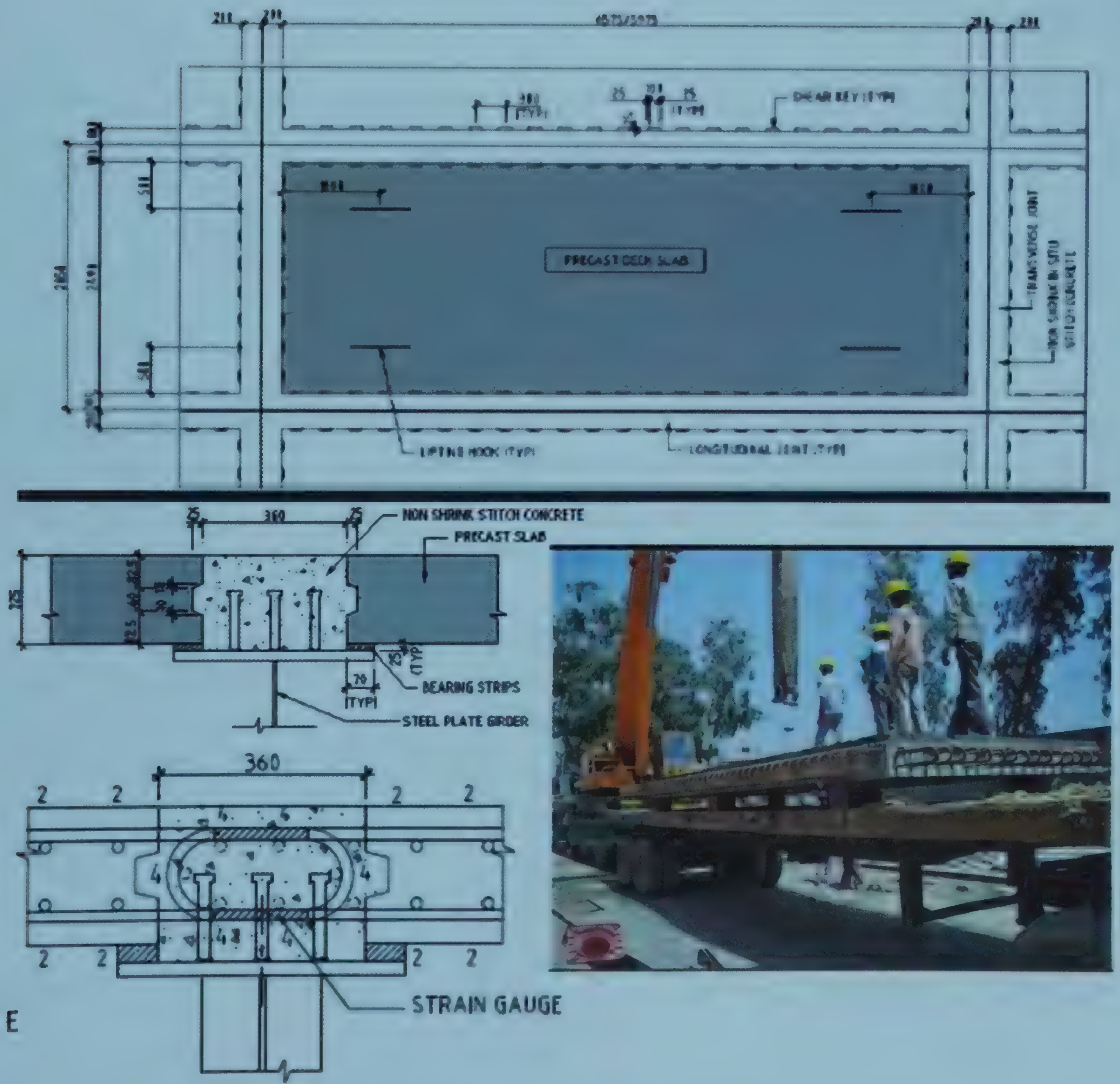

SHEAR KEY (TYP)
PRECAST DECK SLAB
TRANSVERSE JOINT
NON SHRINK IN SITU STITCH CONCRETE
LIFTING HOOK (TYP)
LONGITUDINAL JOINT (TYP)
25
360
25
NON SHRINK STITCH CONCRETE
PRECAST SLAB
225
BEARING STRIPS
(TYP)
STEEL PLATE GIRDER
360
2
2
2
2
2
2
2
2
STRAIN GAUGE
E

Fig. 4.12 (b) Composite Deck Structure: Signature Bridge

When a single plane of stay cables are provided at the centerline of the deck, **Fig. 4.5** (Chambal) the stay cable system will assist in transferring vertical loads only, while eccentric loads on the deck (which cannot be handled by the single plane of stay cables) a box girder section capable of resisting torsion becomes necessary. For longer span applications though, an aerodynamically designed box section is generally deployed, (**Chapter 9**).

In both the cases discussed above, i.e. single or two planes of stay cables, the deck section must be capable of spanning between the supports provided by the stay cables. Additionally, the deck must have the capability to resist compression caused by the inclination of the so-called 'self-anchored' stays, **Fig. 4.2**.

Second order effects (P-Delta) become important when the deck is thin. The calculations of the second order moments are complex due to the presence of stay cables which provide discrete elastic supports to the deck, **Fig. 4.2**, and the fact that the axial compressions get accumulated and are at a maximum near the Pylons. Svensson in **Para 4.1.4.2** of his book **[1]** explains how a simplification can be made by considering the deck as a 'beam on elastic foundation' with a constant subgrade modulus subjected to a constant axial force over the whole length.

4.3 The Pylon

The main purpose of the pylon is to transfer the loads imposed on it by the stay cables down to the top of the foundation. It also has to restrain the deck for lateral loads in the transverse direction. Since the vertical loads of the deck are catered for by the stay cables there is no need for the deck to be supported directly on the deck, **Fig. 4.2**. When a single plane of stays supports the deck the pylon is located on the median verge of the deck, ie, in the plane of the stays, **Fig. 4.13** (Chambal). The pylon is a straight-forward column cantilevering out from the foundation. However, for a laterally supported deck with two planes of stay cables the shape becomes complicated **Fig. 4.10** (Naini Bridge), **Fig. 4.11** (Atal Bridge). Being a compression member, it is almost always in concrete unless there is a special reason to make it in steel.

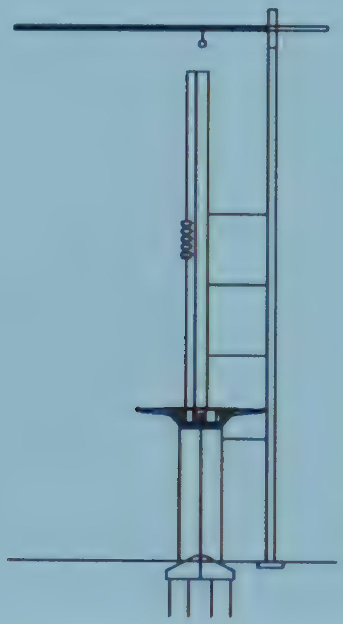


Fig. 4.13. Chambal Bridge Pylon: 80 m Tall Pylon With Single Plane of Stays Erection by Crane

The ratio of the pylon height above the bridge deck to the main span length is typically of the order of 1:5. This can be derived from the consideration that the stay cables whose main purpose is to carry vertical load are not made flatter than say 27° as otherwise they would become practically ineffective for the main purpose for which they have been provided. Incidentally, flatter stays can be provided, in which case they would act more like 'external prestress'. Such bridges are called Extradosed Bridges in which the intention is to carry only some part of the vertical loads by stays while the remaining would be carried by the deck in flexure.

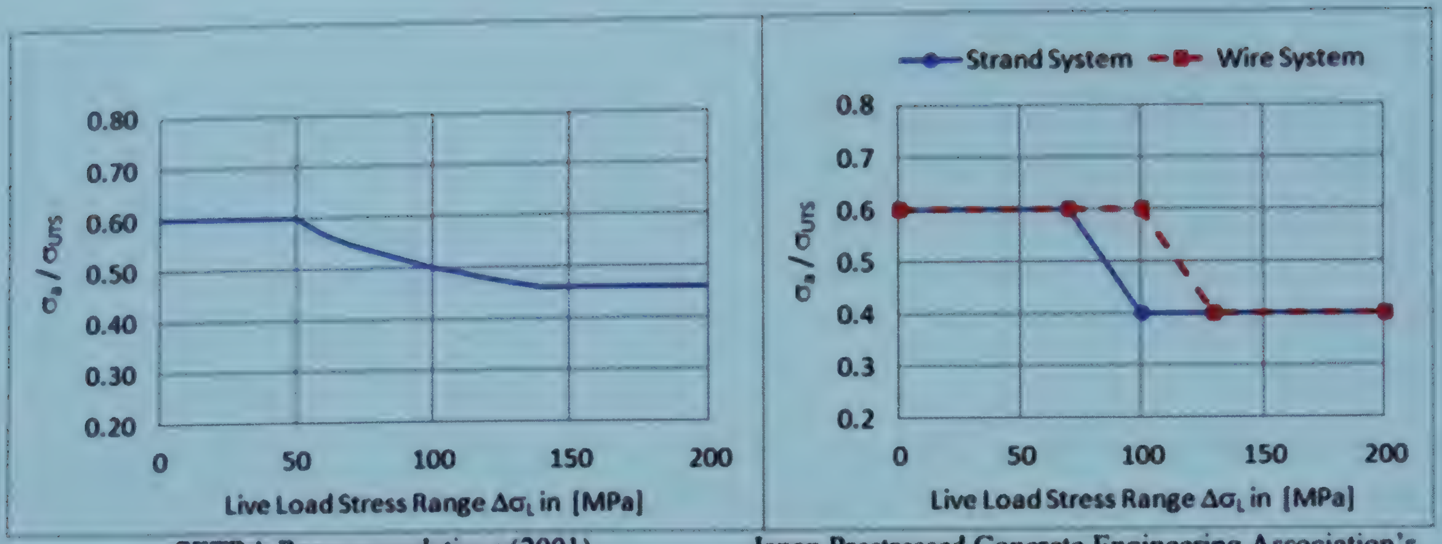
Due to the great height of the pylon its buckling and second order effects need to be paid special attention and so also the wind effects (**Chapter 9**). The stiffness of the pylon is an important consideration for reducing the sagging effects in the main span. As already discussed, the stiffness of the pylon in a bridge with only one pylon which supports one span each on either side is exceedingly important and an " A" shape becomes appropriate **Fig. 4.7** Worli Bandra.

4.4 The Stay Cables

4.4.1 *Fatigue Strength*

The stay cables which support the deck are anchored to the deck as well as to the pylon at different heights. These are made up of high tensile steel wires. Modern day bridges use the parallel strands or parallel wires **(Chapter 10),** which offer the advantage of greater extensional modulus compared to twisted wire strands or 'locked coil' ropes, and, thus reduced girder deflections and flexure. Cables can have a "harp" or "fan" arrangement or they can be an "intermediate" system as already described in **Chapter 3**. The spacing of cables has a profound overall influence upon the design of the bridge. A small spacing, like the one used in modern day cable stayed bridges, leads to a lighter deck girder. Furthermore, this choice results in the cable sizes being smaller and more easily manageable in transportation as well as in construction.

The design of Stay cables are primarily based on the criteria of fatigue. The axial fatigue strength of stay cables is deemed to be acceptable at 2 million cycles for usual cases. Fatigue is dependent on the change of stress in stays resulting from the application of live loads and the permissible upper limits i.e. 50% UTS for Cable Stayed Bridges and 60% UTS for Extradosed Bridge cables as specified in fib 89 and slightly different values for SETRA **[8]** and the Japanese Codes **[9]** as shown in **Fig. 4.14**. As can be seen, it is the change in stress in the stay cable due to live load which decides the permissible stress in the stay as a proportion of its ultimate load capacity. This fact can be advantageously used to determine the sectional area required in each stay cable individually instead of having a fixed value for the whole bridge, **Fig. 4.23**.



SETRA Recommendations (2001)

Japan Prestressed Concrete Engineering Association's Specifications [Ogawa and Kasuga 1998] [Kasuga 2006]

Stress range due to live loads decides fatigue

CABLE STAY BRIDGE ~ 0.4 Ultimate Strength
EXTRADOSED BRIDGE ~0.6 Ultimate Strength

Fig. 4.14 Fatigue of Stay Cables is the Determining Factor for Allowable Stresses [8,9]

4.4.2 *The Back Stay Cable*

For balancing the permanent loads it is advisable that the horizontal component of the stay forces in the main span are in equilibrium with the horizontal component of the stay forces in the side span thereby avoiding bending moments in the pylons.

Live Loads on the main span in a typical 3 span configuration of the bridge, **Fig. 4.3,** the pylon is pulled towards the main span by the stays supporting it. These forces are counteracted by the tension in the stays connected to the side spans. The back stay cable is unique in the sense that it is connected to the deck at a point where there is no vertical deflection because of the anchor or hold-down piers that prevent uplifting of the deck.

The behavior of the Back Stay Cable with the Live Load in the Main Span and Side Spans respectively is shown in **Fig. 4.15**. It has the largest variation of stress amongst all stays as tension has the highest value in the first case while the second case induces compression.

The largest horizontal component inducing compression in the deck also is due to the backstay cables, **Fig. 4.15**.

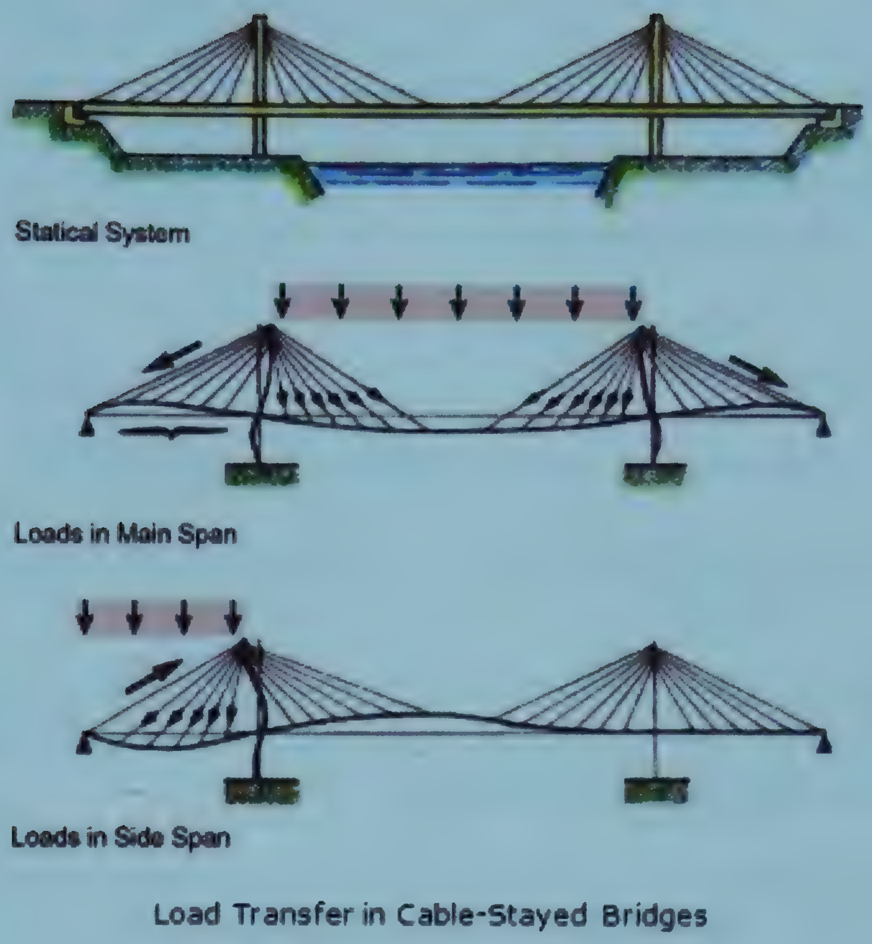


Fig. 4.15 Behavior and Role of Backstay Cable SVENSSON

4.4.3 *Seismic and Wind Effects*

Seismic and aerodynamics have contradictory demands on the structure. For seismic design a more flexible bridge will have less response. On the other hand, for aerodynamic stability a stiffer structure is preferable. Some compromise between these two demands is required. A device that connects the girder and the tower, which can break at a certain predetermined force will help in both events. Under aerodynamic actions, it will suppress the onset of the vibrations as the connection makes the structure stiffer. Under seismic load, the connection breaks at the predetermined load and the structure becomes more flexible, thus reducing the fundamental frequency of the bridge. Detailed coverage of these concepts are available in **Chapters 8 and 9**.

4.5 Steps for Preliminary Design

Preliminary design of a cable stayed bridge is essentially an iterative process, just as it is for any structure. The number of cycles of iteration depend upon the past experience of the designer, and, the back-up information available to him through standards and other related literature. As a first step, the general proportions of the structure relative to the span are to be assigned on the basis of guidelines discussed in the earlier part of this chapter. Next, the sizes of the various elements have to be assumed (estimated). It is clear that all the three major elements of a cable stayed bridge, namely, the deck, pylons and cables interact with each other in this highly indeterminate system, and cannot therefore be designed independently of each other. In this case nevertheless, one may begin with the deck and the cables. A good strategy is to take starting values based on data from similar (or nearly similar) existing bridges.

Since the stay cables are closely spaced, the central idea is that the deck should remain horizontal for permanent loads. The bridge deck can then be designed such that it behaves, like a continuous beam supported on rigid supports, the location of which are at the cable anchorage. Not only are the Bending moments in the deck minimized due to permanent loads but so also the creep effects due to bending of the deck. Of course the creep effects due to the axial compression induced by the stay cables would still take place.

The steps recommended are as follows:

Step 1: Select span arrangement, bridge type and configuration.

Step 2: Select Cable Arrangement Type; one plane /two plane of stays (several examples shown in **Fig. 4.5** and **Fig. 4.10.**

Step 3: Select appropriate Section of the deck (one plane stays will generally require box girder section **Fig. 4.5** and estimate average depth of concrete for the section **Fig. 4.16 (a).** Calculate Sectional Properties.

Step 4: Evaluate first estimate of total stay quantity by assuming the value of β **Fig. 4.18** and spacing and thereby designate the area supported by each stay cable.

Step 5 : Calculate DL effects in service condition.

Step 6 : Calculate Live load effects in service condition.

*Step 7: R*efine the value β and re-do the steps 5 and 6 if necessary. This completes the vertical load effects on the bridge.

Step 8: With the structural model so determined one can get into Seismic and Wind Effects if so required in the Preliminary Design stage. Further processing where necessary at the Preliminary Design stage would involve partial load factors and load combinations (in accordance with IRC:6 or as modified in this document) since no provisions exist in the country specifically for Cable Stayed Bridges. Other Chapters in this document may be referred for an exposition of these ideas.

4.6 Preliminary Dimensioning and Estimation of Quantities

4.6.1 *Concrete Deck*

Mermigas **[13]** has published an interesting paper on the quantities of concrete (m^3/ m^2, or average thickness of concrete in the deck) for Cantilever, Extradosed and Cable Stayed Bridges based on previous projects. The information is shown in a concise manner in **Fig. 4.16 (a) & 4.16 (b)** and can be used conveniently in the Concept Design Stage as a starting point for self weight of the deck concrete.

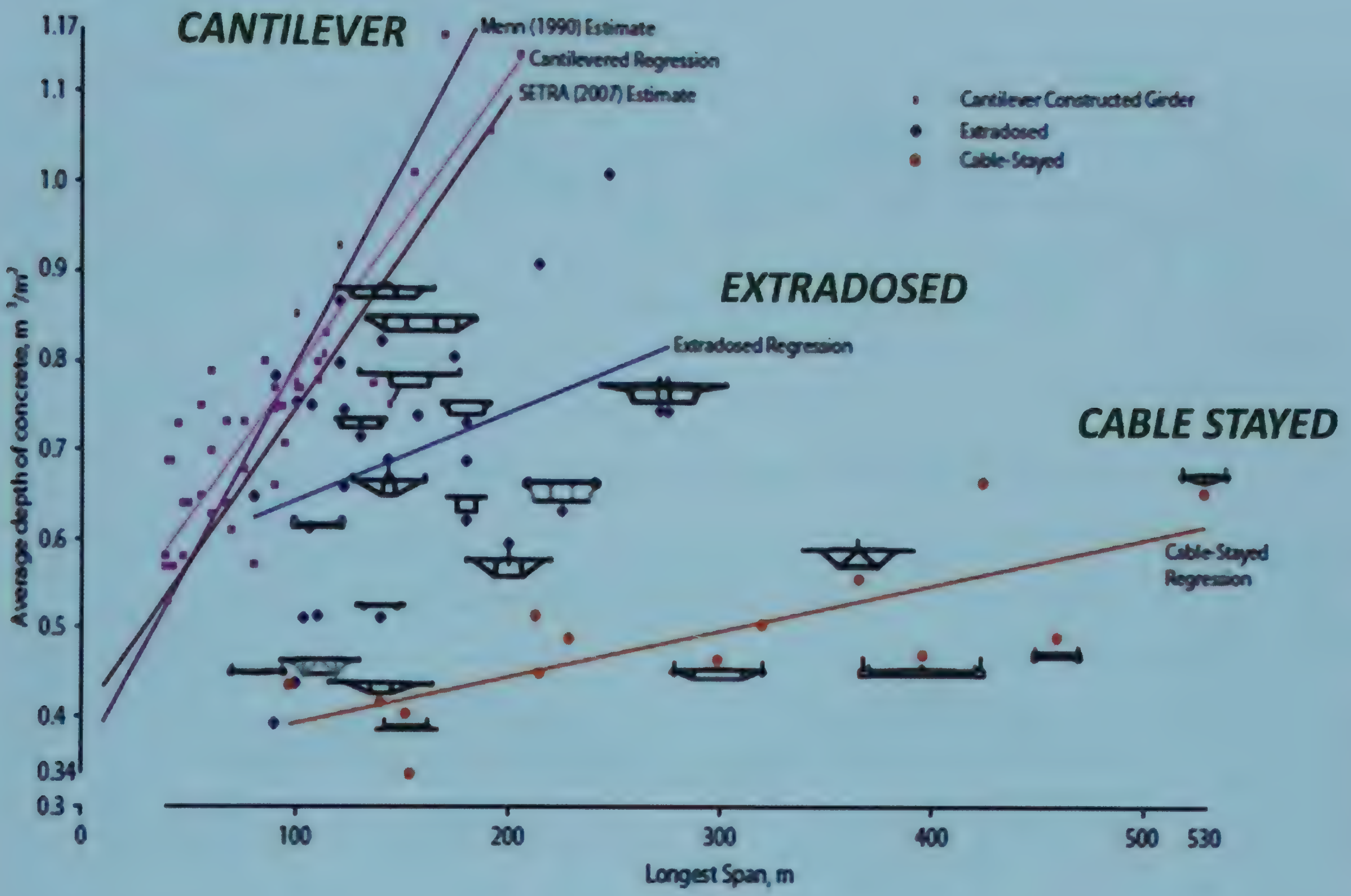


Fig 4.16 (a) Evaluating Approx V.G. Average Concrete thickness of Deck. Mermigas [13]

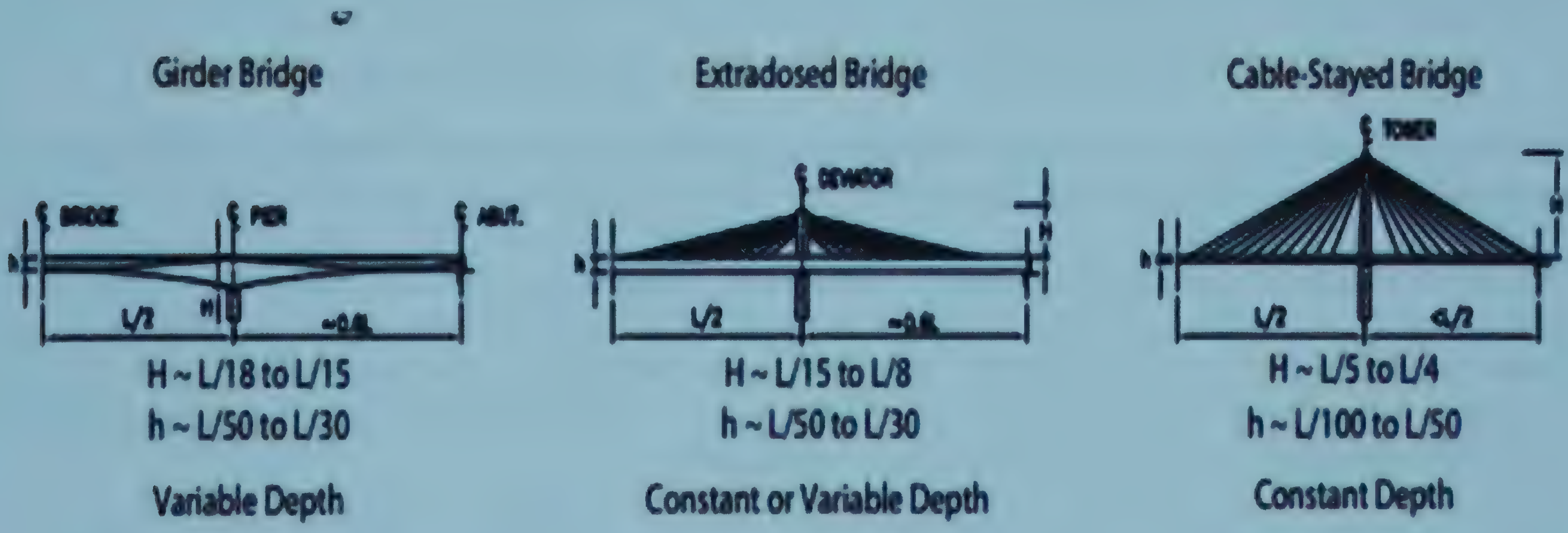

Girder Bridge
℄ BRIDGE
℄ PIER
℄ ABUT.
h
H
L/2
~0.8L
H ~ L/18 to L/15
h ~ L/50 to L/30
Variable Depth
Extradosed Bridge
℄ DEVIATOR
h
H
L/2
~0.8L
H ~ L/15 to L/8
h ~ L/50 to L/30
Constant or Variable Depth
Cable-Stayed Bridge
℄ TOWER
h
H
L/2
<L/2
H ~ L/5 to L/4
h ~ L/100 to L/50
Constant Depth

Fig 4.16 (b) Dimensioning the Superstructure of the Three Types of Bridges

As an illustration the Naini Bridge with 630 m (c/c of Expansion Joints) long module containing the main span of 260 m had an average thickness of 0.62 m including the thickened portion of the deck to prevent uplift at the bearings of the end pier as indicated in **Fig. 4.17 (a) & 4.17 (b).** To increase the factor of safety against uplift the weight of the approach span was also mobilized. The arrangement of stays with respect to the pylon was symmetrical.

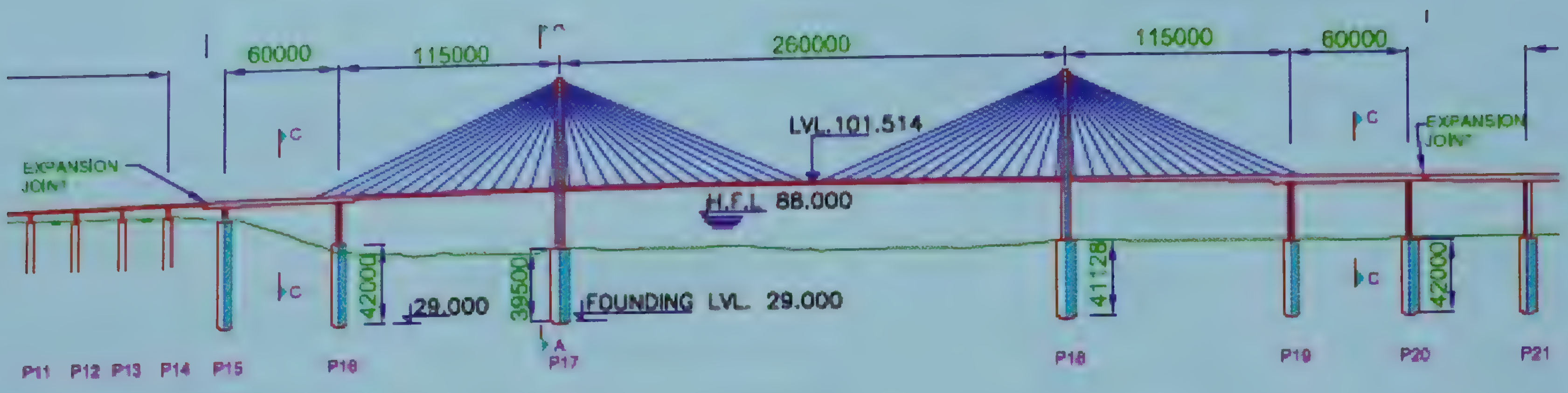

60000
115000
260000
115000
60000
LVL.101.514
H.F.L. 88.000
EXPANSION JOINT
EXPANSION JOINT
42000
29.000
39500
FOUNDING LVL. 29.000
41128
42000
P11
P12
P13
P14
P15
P16
P17
P18
P19
P20
P21

Fig. 4.17 (a) Naini Bridge Module with 260 m Main Span

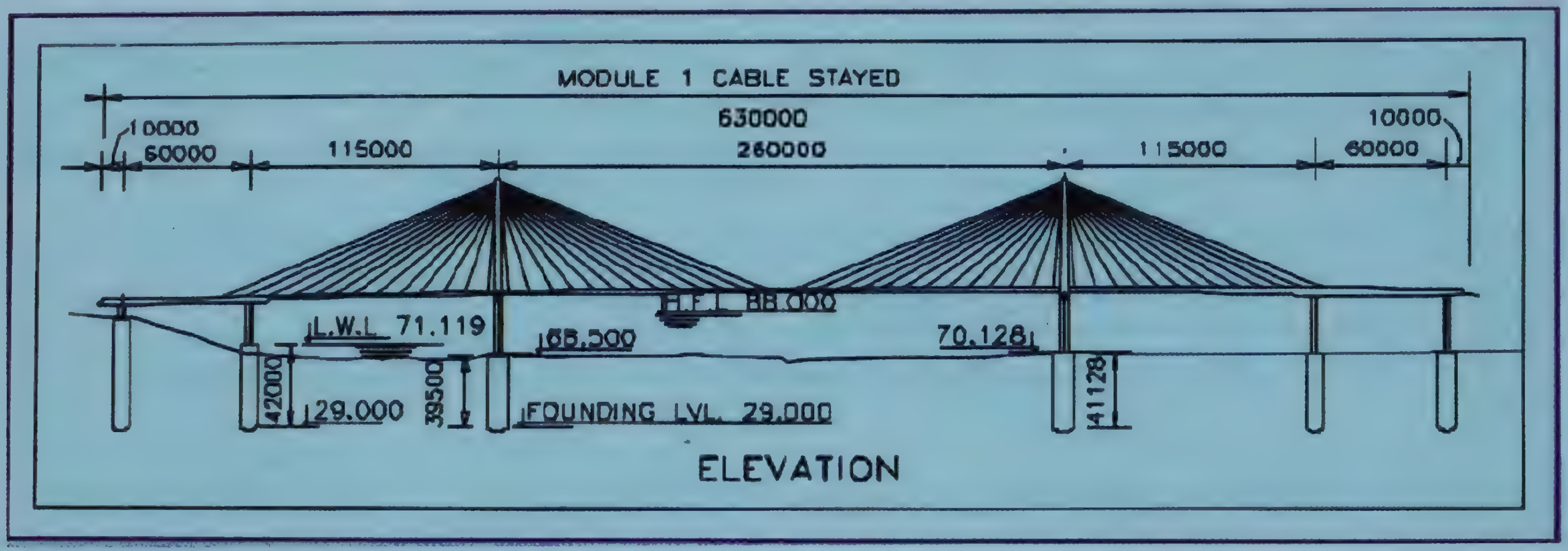

MODULE 1 CABLE STAYED
10000
60000
630000
115000
260000
115000
10000
60000
L.W.L. 71.119
H.F.L. 88.000
68.500
70.128
42000
29.000
39500
FOUNDING LVL. 29.000
41128
ELEVATION

Fig 4.17 (b) Naini Bridge: General Arrangement.

Note that Reactions of Approach Spans act at the two ends of Module 1 which are also the locations of the expansion joints.

The 630 m long continuous bridge deck has fixed bearings on pier P I5 and one directional sliding bearings on the piers P 16, P 19 and P 20. The bridge deck is not supported on the lower cross beam of the pylons P 17, P 18. Only the transverse horizontal forces can be transferred directly from the bridge deck to the pylons. Expansion Joint capacity at P 20A is 1200 mm (Modular Strip Seal with 15 nos. modules)

4.6.2 *Stay Cables*

In a recent publication **[11]** Akio Kasuga has simplified the quantity estimation of stay cables for Extradosed Bridges and Cable Stayed bridges (in Concrete or steel). The ideas presented below have been taken from references 10 & 11. Ogawa and Kasuga in their in 1998 **[14]** first introduced the concept of the Beta factor which is defined as follows:

$$\beta = \frac{\text{Load carried by stay cables}}{\text{Total vertical load}} \times 100\%$$

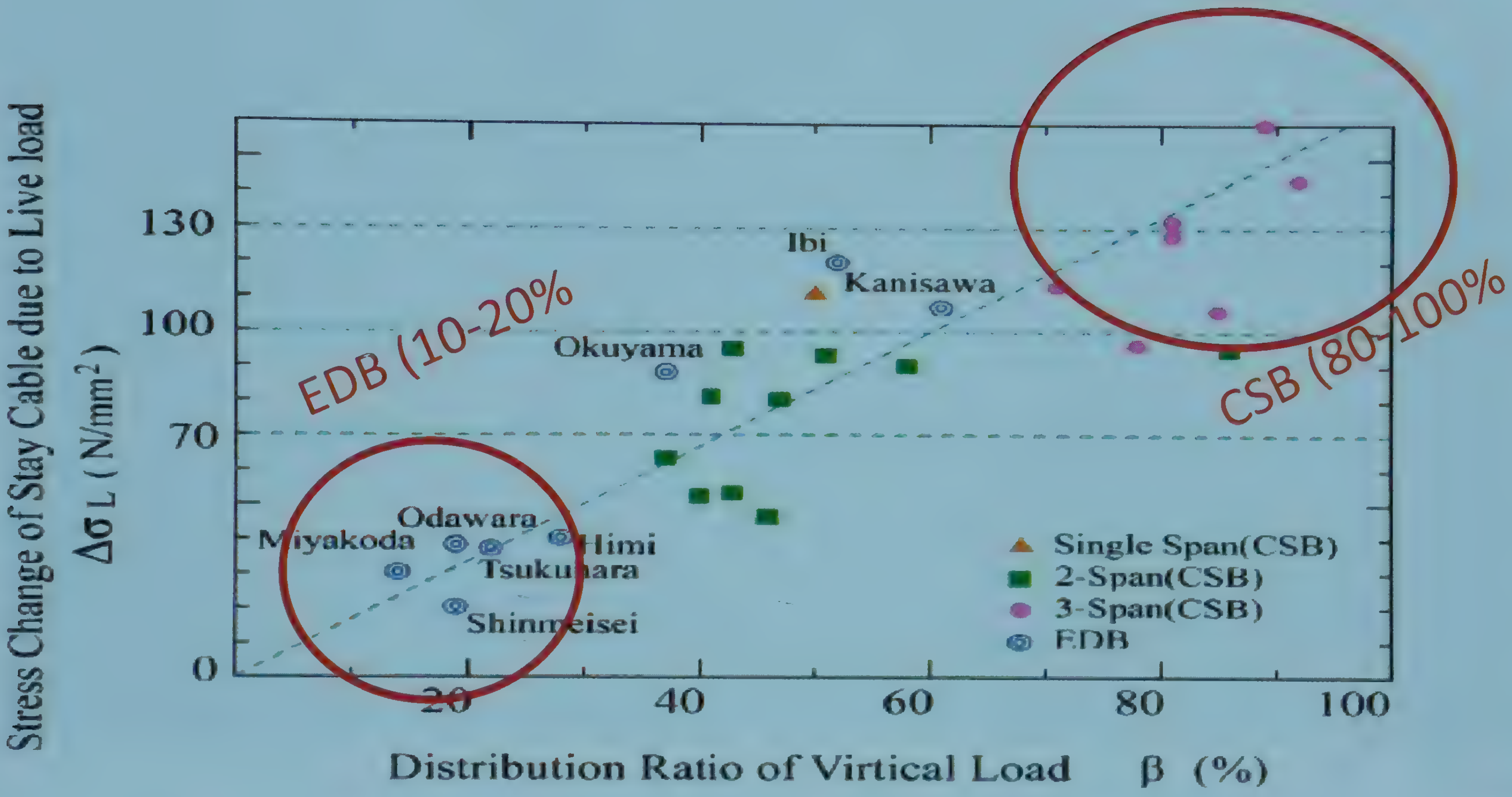


Fig. 4.18 Concept of Beta Factor applied to CSB and EDB [10,14]

From past projects the Cable Stayed Bridges and Extradosed Bridges generally lie in the range shown in **Fig. 4.18**. For cable-stayed bridges, β is in the region of 80 - 100%, as the cables carry nearly all the live load. In contrast, for extradosed bridges, β is in the region of 10 - 20%.

Kasuga **[11]** concludes that the cable-stayed bridge and extradosed bridge can be considered as a continuum rather than having a clear borderline between these two types of bridges, **Fig. 4.18**, with the main girder carrying most of the live load in the latter case. From this point of view the Japanese code has a great advantage as it defines the allowable stress in terms of stress variation (due to live load). It enables the designer to consider the limit separately for each individual cable. In other words, the allowable stress can differ from cable to cable.

This permits the adoption of efficient designs that were not possible under conventional standards that determined a single allowable stress for the whole bridge, be it cable stayed (concrete/steel) or extradosed. However, fib 89 **[7]** on which these Guidelines are largely based mention this idea as 'International Design Practice' without prescribing to it and also mention that such an approach would require the redefining of the limits of permissible service stresses in the cables depending on the actual levels of variable stresses.

In the recently published IABSE Bulletin 17 **[12]** the approximate range of values have been summarized for Stay cables, Extradosed cables and PT Tendons as follows:

Table 4.1 Live Load and Dynamic Characteristics of Cable Types

Cable type	Live load stress change $\Delta\sigma_L$ (MPa)	Approx max allowable stress
Stay cables	~100 ($60 < \Delta\sigma_L < 160$)	40–45% GUTS
Extradosed cables	~50 ($30 < \Delta\sigma_L < 100$)	Dependent on $\Delta\sigma_L$
PT tendons	~15 MPa	70–75% GUTS

4.7 Multi span cable stayed bridges

Multi span cable stayed bridges have to be dimensioned a little differently from the 3-span construction which largely formed the basis of discussion so far.

The tower height should be chosen so that the flattest cables have the usual inclination of about 27°. This gives a ratio of tower height to each main span of about 0.25.

If the spacing of the towers is kept constant, **Fig. 4.19 (a)** the permanent loads will be balanced which is of great benefit. However, the live loads create larger effects as compared to the 3-span construction due to the absence of short side spans, **Fig. 4.20**.

The Mandovi Bridge in Goa, **Fig. 4.19 (a) & 4.19 (b)** is currently the only multispan cable stayed bridge in the country, with a span arrangement of 85+3x150+85 m, making a total length = 620 m between abutments where the expansion joints are located. **Fig 4.19 (c)** shows the Cross Section with a total width of Bridge Deck as 21 m for 4 lanes of traffic plus 3 m median for accommodating the stay cables.

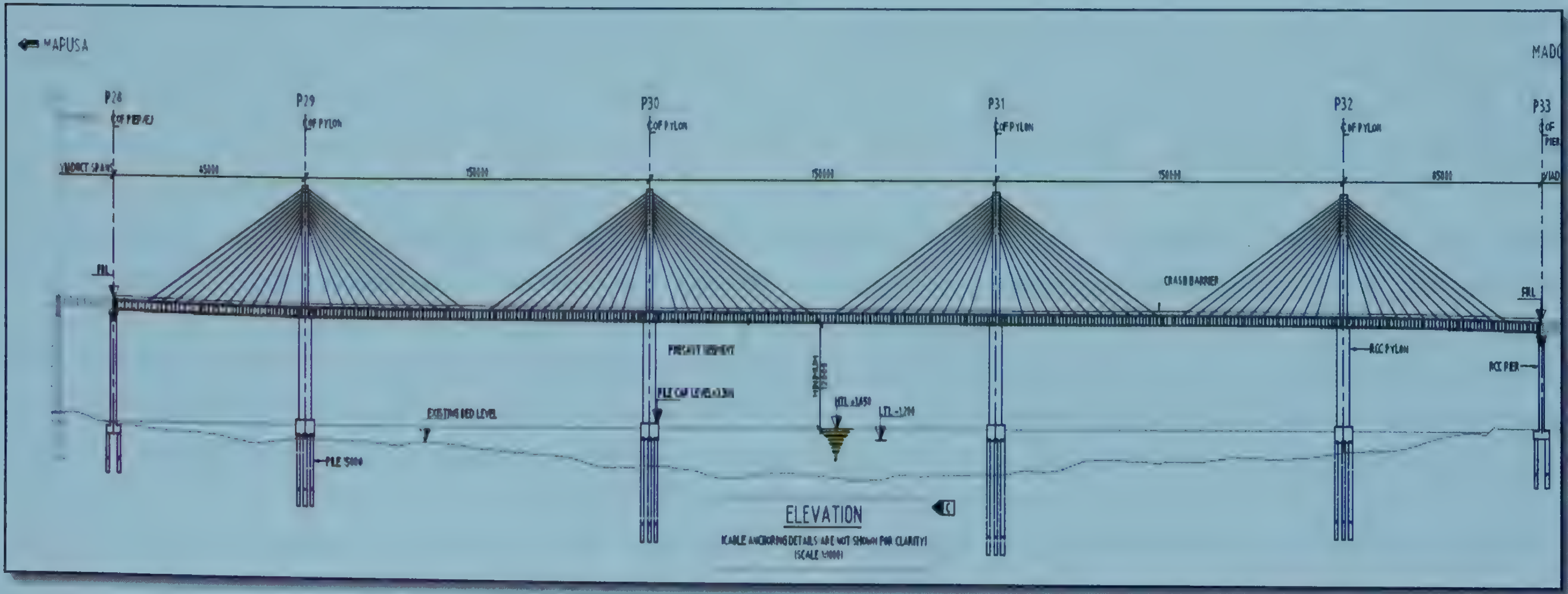

MAPUSA
MADO
P28
P29
P30
P31
P32
P33
OF PYLON
VIADUCT SPANS
85000
150000
150000
150000
150000
85000
FRL
CRASH BARRIER
PRECAST SEGMENT
PILE CAP LEVEL
EXISTING BED LEVEL
PILE
HTL
LTL
RCC PYLON
RCC PIER
ELEVATION
(CABLE ANCHORING DETAILS ARE NOT SHOWN FOR CLARITY)
(SCALE 1:1000)

Fig. 4.19 (a) Mandovi Bridge, Goa

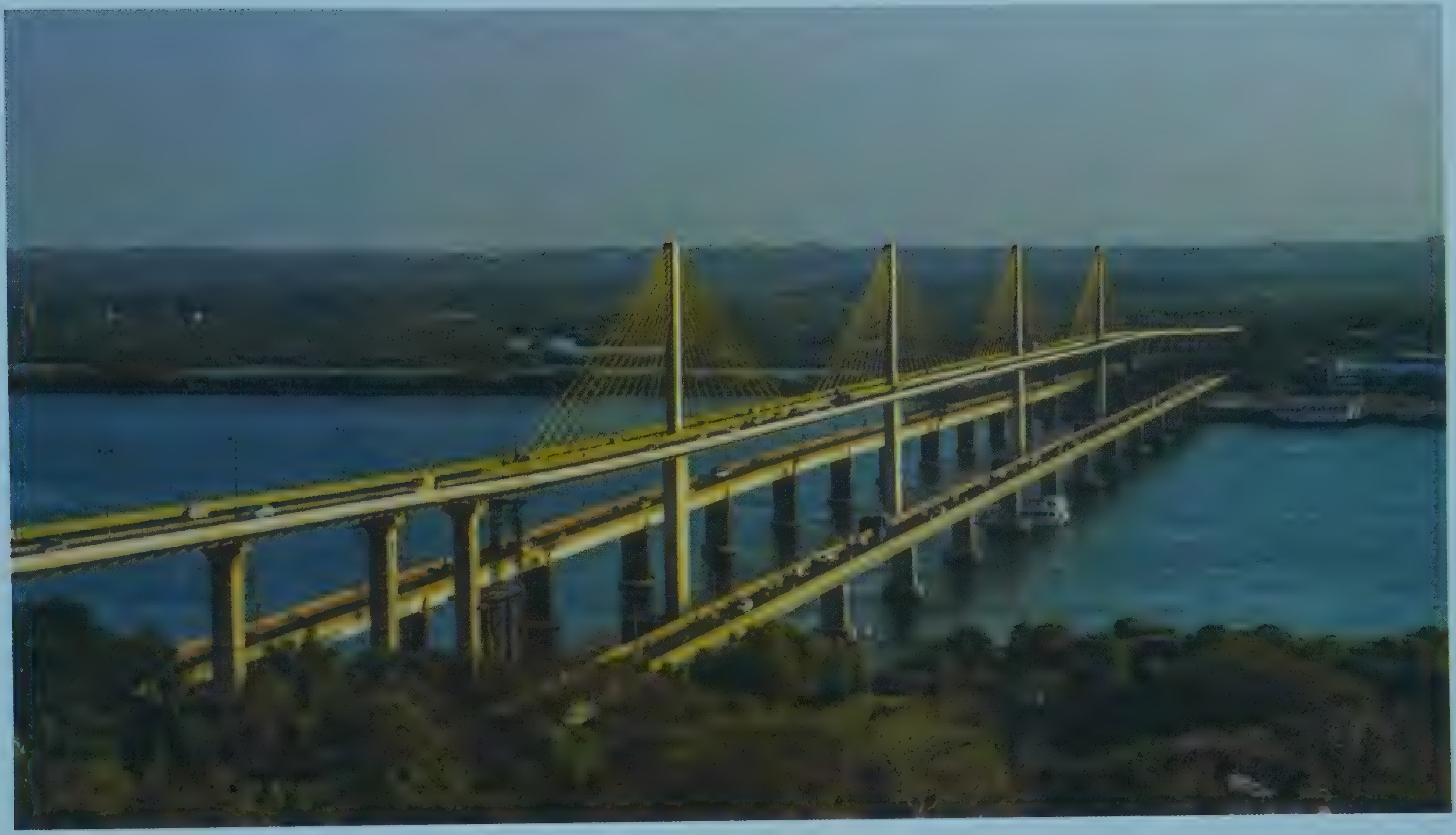


Fig. 4.19 (b) Mandovi Bridge, Goa Third River Crossing

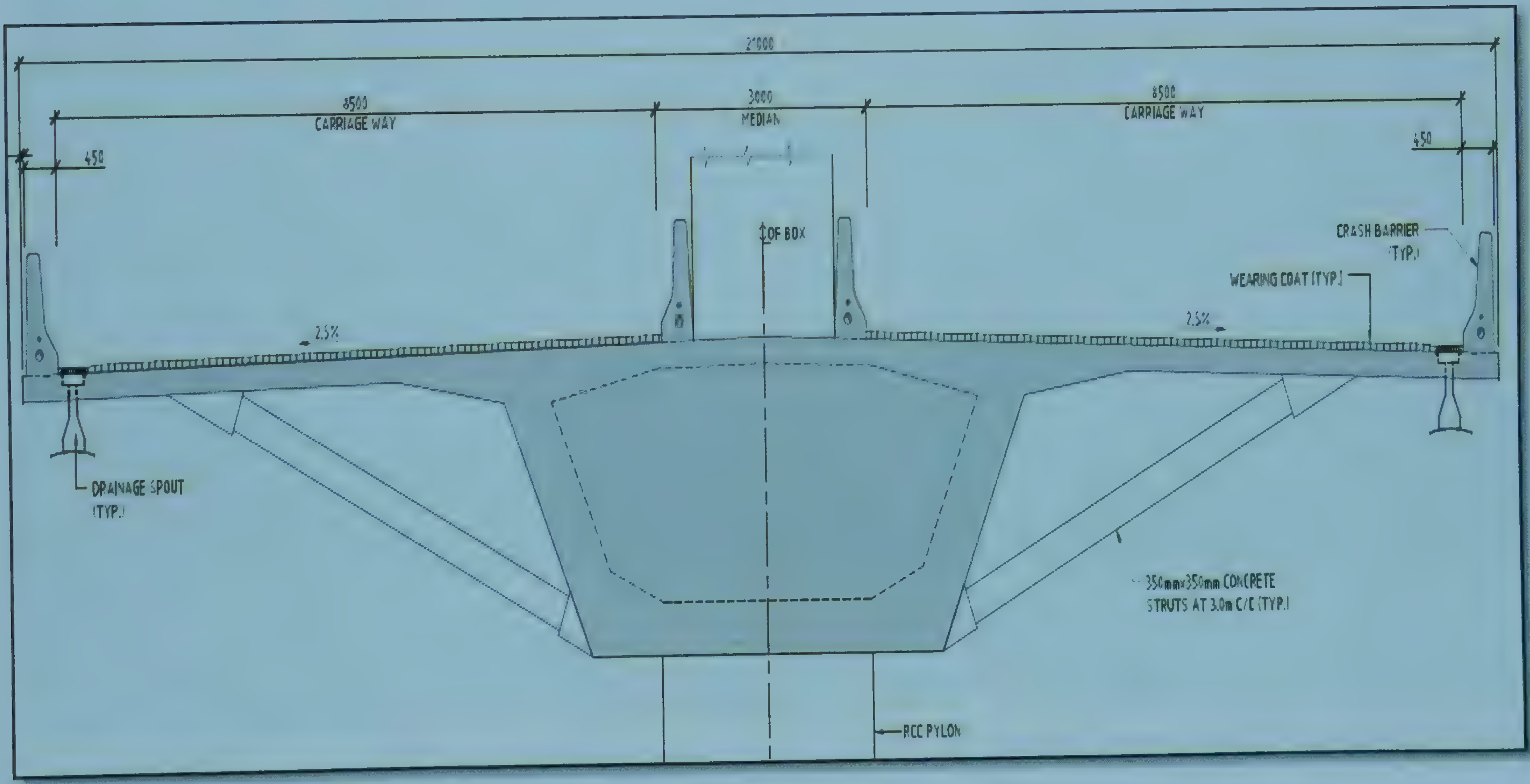

21000
8500
CARRIAGE WAY
3000
MEDIAN
8500
CARRIAGE WAY
450
450
℄ OF BOX
CRASH BARRIER (TYP.)
WEARING COAT (TYP.)
2.5%
2.5%
DRAINAGE SPOUT (TYP.)
350mm×350mm CONCRETE STRUTS AT 3.0m C/C (TYP.)
RCC PYLON

Fig. 4.19 (c) Mandovi Bridge, Goa. Deck Cross Section

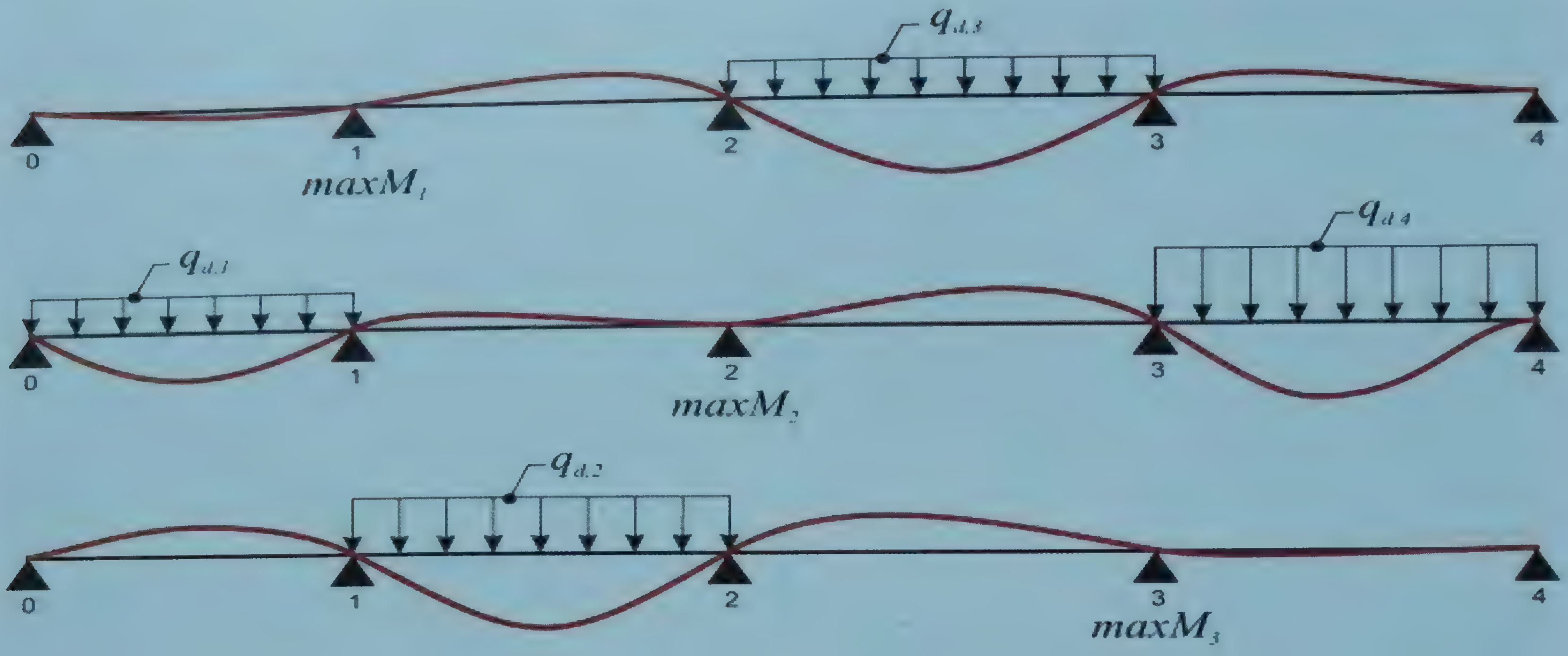


Fig. 4.20 Live Load Effects in Multispan Bridge

4.7.1 *Stabilising the pylon*

In the absence of short side spans when the stay forces on the pylon cannot be balanced, a good option is shown in **Fig. 4.21** Anchor cables from the top of one pylon are attached to the deck at the next pylon location which provides the necessary restraint to the pylon top.

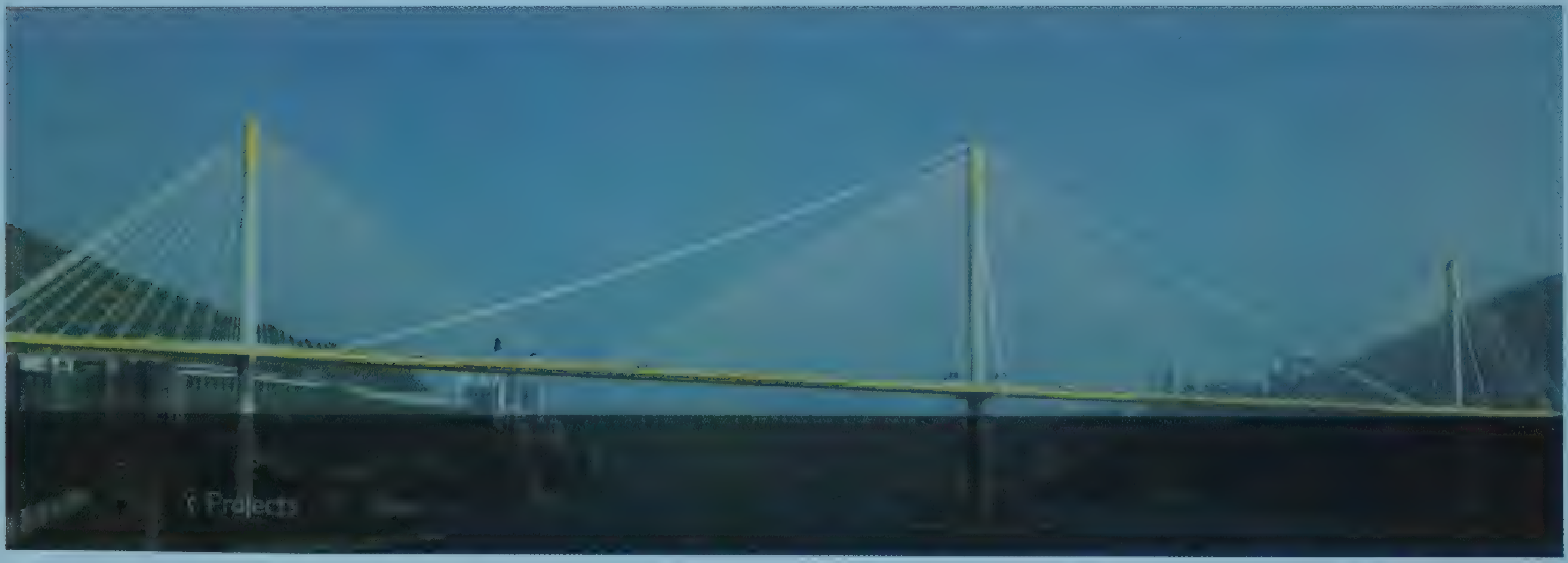


Fig. 4.21 Tin KAU Bridge, Hongkong - stabilising the pylon Concept; Schlaich Bergerman & Partners

4.8 Recent Cable Stayed Bridges

Three recent interesting cable stayed bridges of long span with unique concepts using steel-concrete composite construction require discussion to make this Chapter complete:

- The New Zuari Bridge in Goa (Main Span=360m)
- The Beyt Dwarka Bridge connecting Okha to Beyt Dwarka in Gujarat (Main Span=500 m)
- The Delhi Katra Expressway under construction which incorporates the 700 m long main span over River Beas, which will be the longest in the country when completed.

Brief details of the three bridges are discussed below:

4.8.1 The New Zuari Bridge in Goa (Manohar Setu), 2022

The bridge has been constructed with a cable stayed module of 640 m with span arrangement of 140+360+140 m, **Fig. 4.22**.

As a matter of fact there are two parallel structures constituting the bridge each catering to 4 lanes of traffic with a width of 27m with 10.7m wide separation. The bridge will cater to the traffic between key locations in North and South Goa, reducing the travel time by 30 minutes.

The pylons reach a height of 100 m and are planned to support a spectacular tourist observatory-cum-gallery and restaurant (**Fig 4.23**) which are expected to be a huge tourist attraction.

The deck is fabricated on shore in segments typically of size-27 m*12 m weighing 100 t (**Fig. 4.24)** which are transported on barges and raised in position with the help of segment lifters used for the free cantilevering of the bridge shown in **Fig 4.25**.

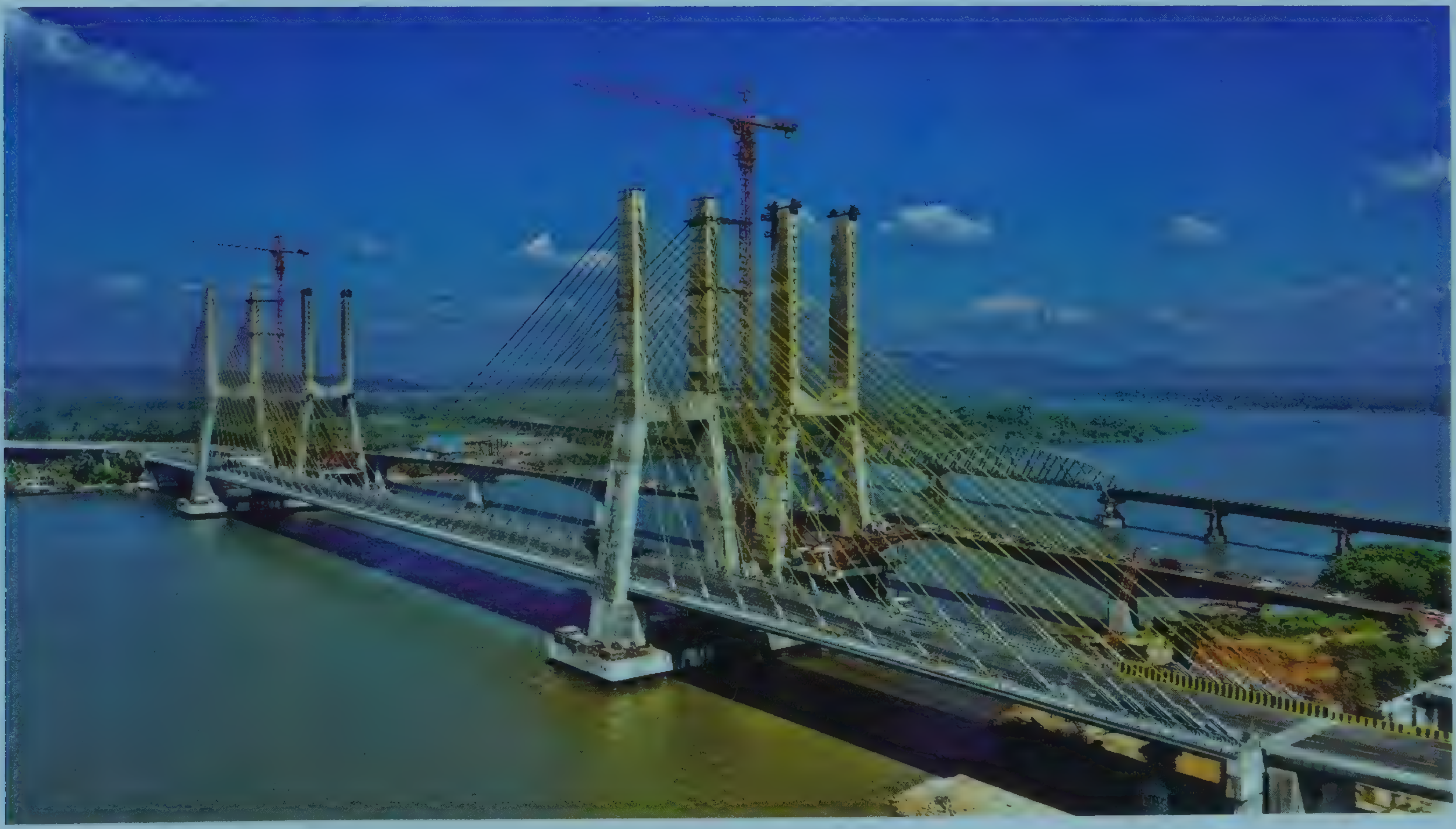


Fig. 4.22 Photo of New Zuari Bridge Completed

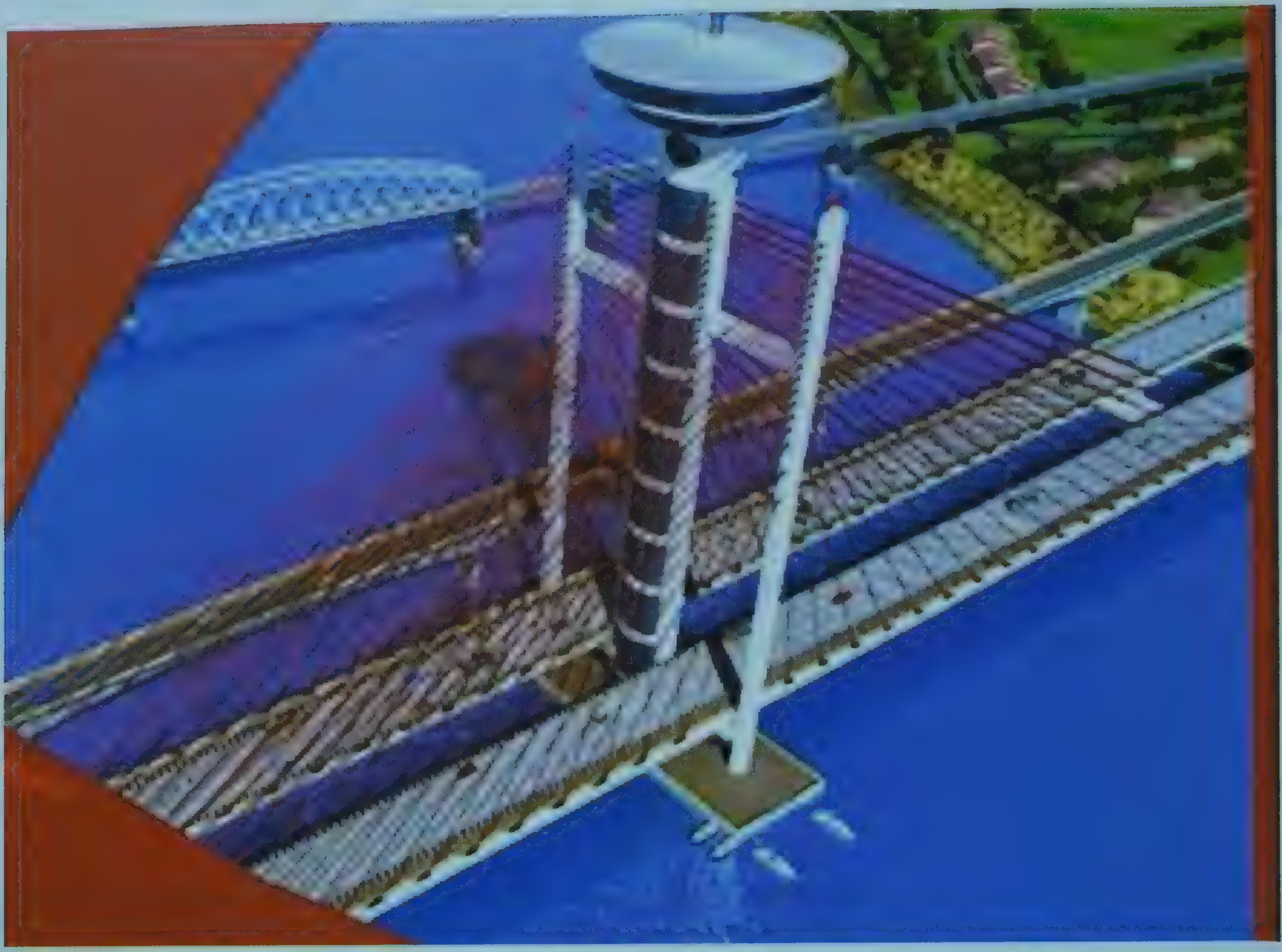


Fig. 4.23 Proposed Viewing Gallery with revolving restaurant

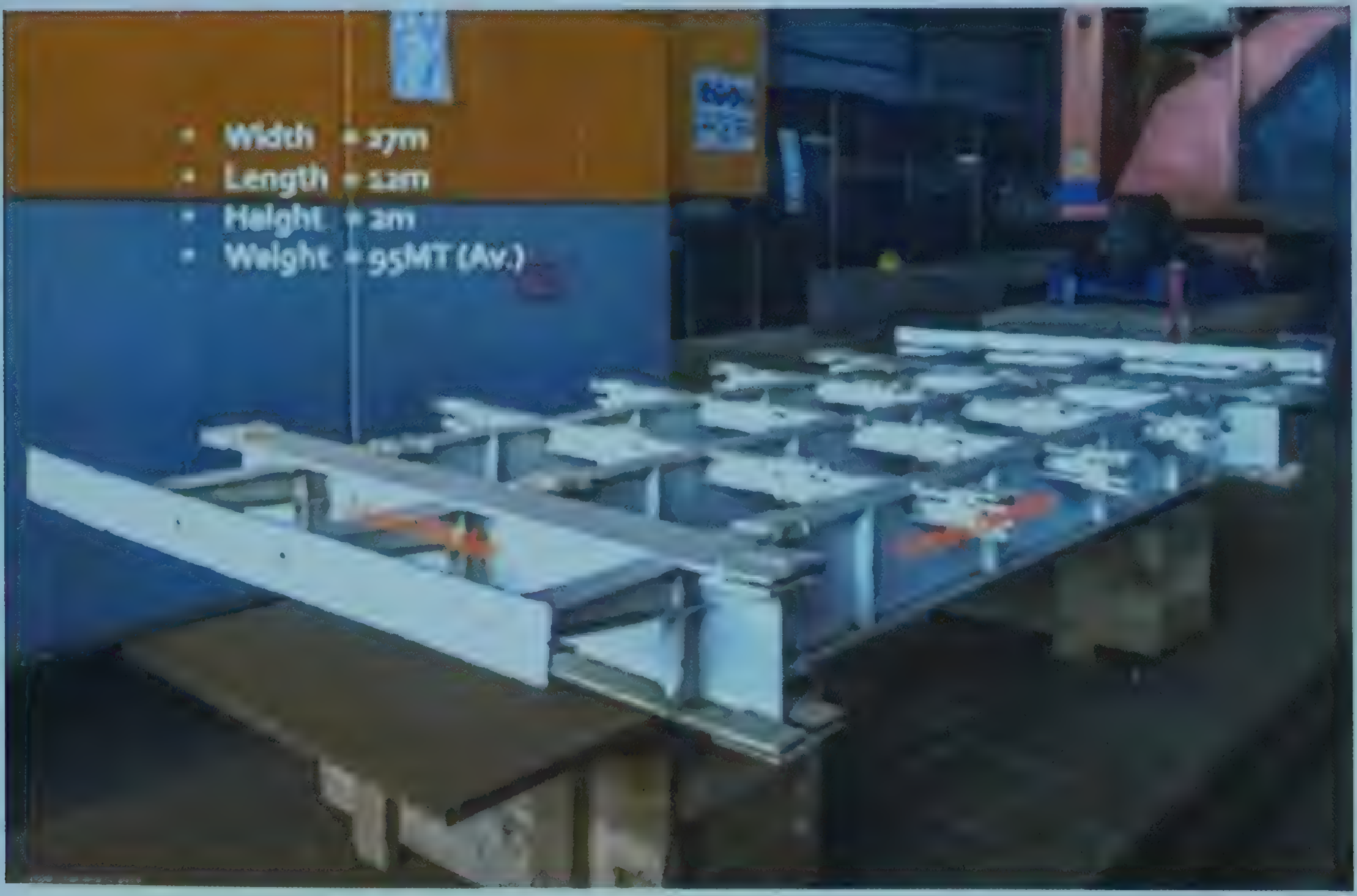

Width = 27m
Length = 12m
Height = 2m
Weight = 95MT (Av.)

Fig 4.24 Steel deck segments fabricated on shore

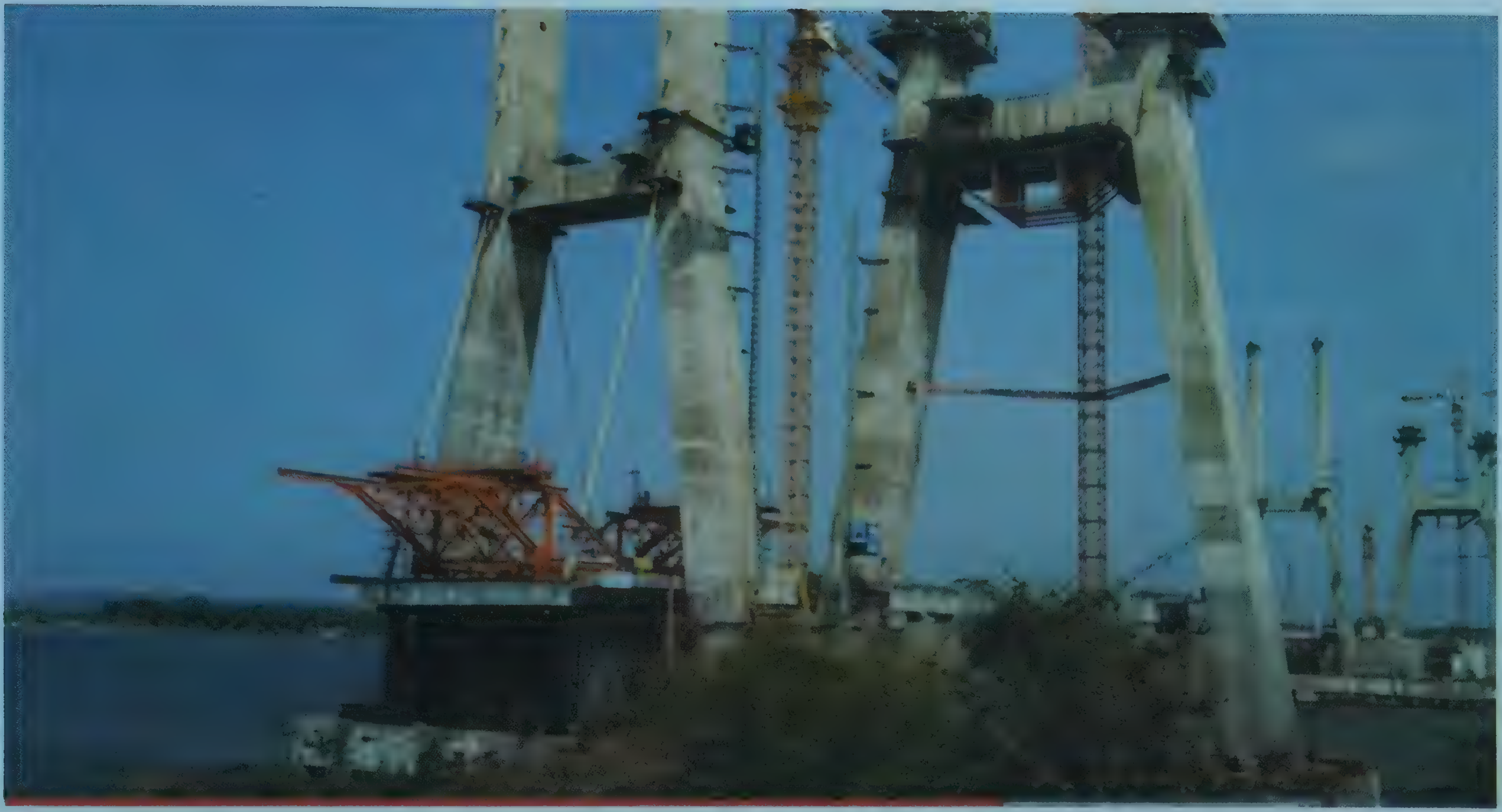


Fig. 4.25 Free Cantilevering of the bridge for the two Decks (Constructed Simultaneously)

4.8.2 Beyt Dwarka Bridge, Gujarat (Sudarshan Setu), 2024

The bridge is proposed with a cable stayed module of 900 m having a span arrangement of 2*50+100+500+100+2*50 (**Fig. 4.26**). The 27.2 m wide deck is constituted from a steel grillage with concrete deck. It caters to 4 lanes of traffic plus 2 x 2.5 m wide footpaths. The approaches and the end spans were constructed first and then the main span was cantilevered out from the pylons. The bridge connects mainland (Okha) to Dwarkadhish Temple, one of the destinations of Char Dham, the pilgrimage circuit.

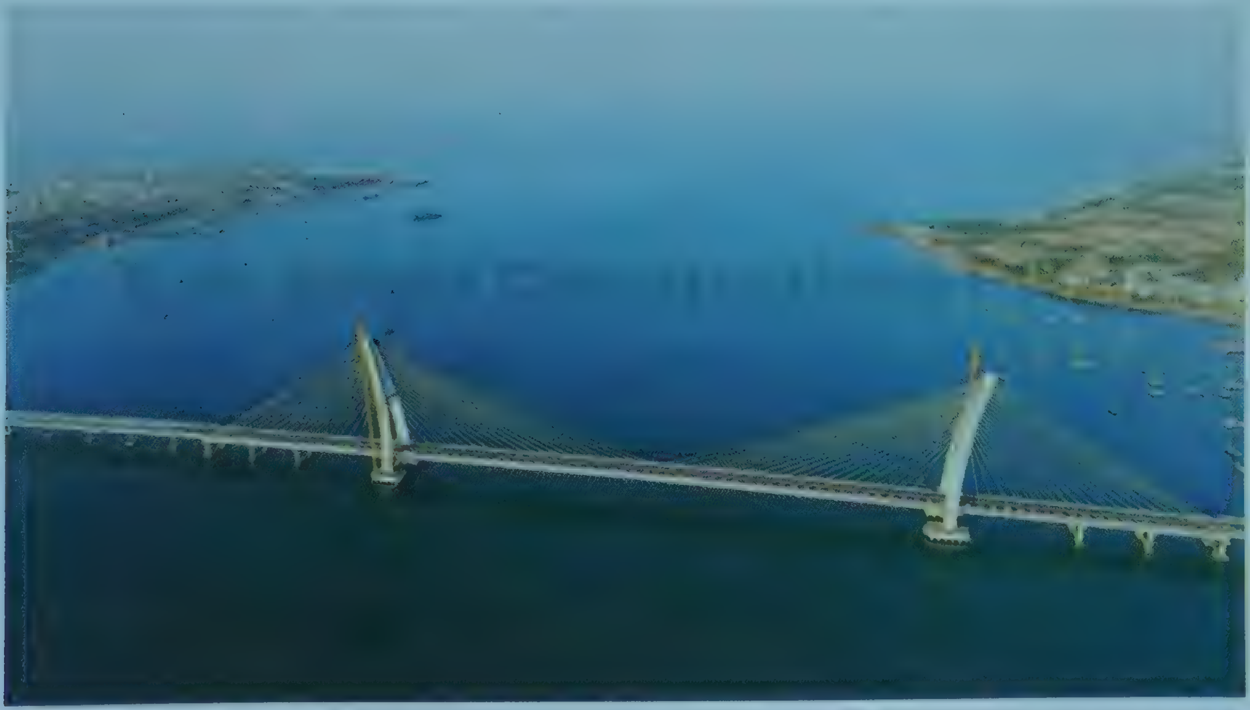


Fig. 4.26 Beyt Dwarka Bridge, Gujarat: has a main span of 500m

4.8.3 Delhi-Katra Expressway Bridge, under Construction

The bridge located on the Expressway is over River Beas **(Fig. 4.27)** to protect the Dolphin Conservation Reserve, with a proposed span arrangement of 300 m+700 m+300 m with a 4 lane configuration, expandable to 8 lanes. The Expressway will boost connectivity and cater to religious tourism, easing access to Vaishno Devi (Katra) and Golden Temple (Amritsar).

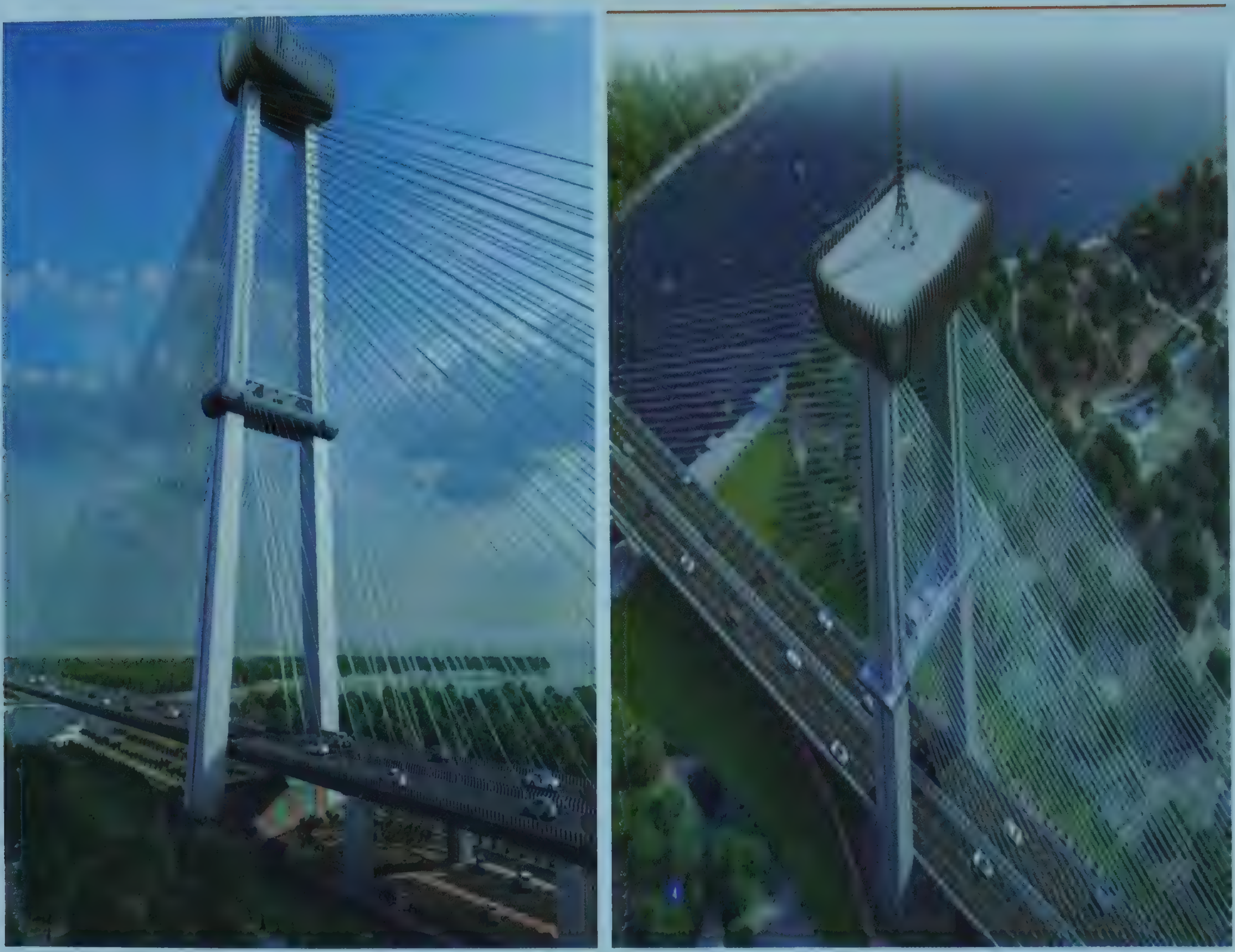


Fig. 4.27 Delhi Katra Expressway: India's Longest Cable Stayed Span : Under construction 6 lanes Span =700 m

CHAPTER - 5

CONSTRUCTION

5.1 General

The cable-stayed bridges covered by this guideline differ from girder bridges & extradosed bridges. However, there are some commonalities between cable-stayed and extradosed bridges, like use of cables, construction of upper pylon, installation and anchoring of cables. Despite these commonalities, as the size of cable stay bridges (Span, pylon heights etc.) are comparatively larger, stress and deformation produced in the members can be significantly influenced by the construction method and sequence adopted

With regards to construction, it is necessary to satisfy required conditions such as environmental effects, construction safety, cost and period, which are based on its relation to entirety, purpose, type, scale and characteristics of project.

The construction planning of cable-stayed bridges shall be prepared prior to the commencement of construction by considering the aspects, that are necessary to satisfy the required performance of structure and requirement of the project.

In addition, aspects about the quality and schedule control also should be examined while planning construction.

In comparison to other bridges, for cable stayed bridges, the influence of construction method, construction sequence and construction accuracy are significant. Therefore, it is important that qualified engineer(s) with sufficient knowledge and experience are constantly stationed at the construction site to provide appropriate construction management.

5.2 Deck Erection and Fabrication

Generally there are three methods of deck erection for cable stay bridges. The staging method of erection is most often used where there is a low clearance requirement to the underside of the structure and temporary bents will not interfere with any traffic below the bridge. Its advantage is accuracy in maintaining required geometry and gradient and its relatively low cost.

The push-launching technique has been used successfully where care must be taken not to interfere with traffic below the bridge and where cantilever construction is impractical. In this method, large sections of bridge deck are pushed out over the piers on rollers or sliding teflon bearings. The deck is pushed out from both abutments toward the center, or, in some instances, from one abutment all the way to the other abutment in which case it is called progressive placement method. Assembling the components in an erection yard at one or both ends of the structure and progressively pushing the components out into the span as they are completed, can simplify construction and reduce costs. With this method as much as 1500 t of steel, spanning a number of supports, have been pushed out and, in some instances, the method has been used where a horizontal curvature is required.

The cantilever erection method is very often employed in cable-stayed bridge construction where temporary supports are uneconomical or not applicable. In some cases, it may increase the steel requirements in terms of temporary post-tensioning bars, permanent pre-stressing or steel deck quantity over that required for final positioning to accommodate the increased moments and shears during the erection process. The principal advantage is that it does not interfere with traffic below the bridge.

5.2.1 *Staging method*

In cable stayed bridges due to the inclination of cables, the load-carrying capacity of the single cable depends on the transfer of the horizontal component of the cable force through the deck. Therefore, a given cable cannot be installed before the adjoining part of the deck has been erected. Then the easiest way of erection is to support the deck over temporary supports before the cables are stressed as shown in the **Fig. 5.1**.

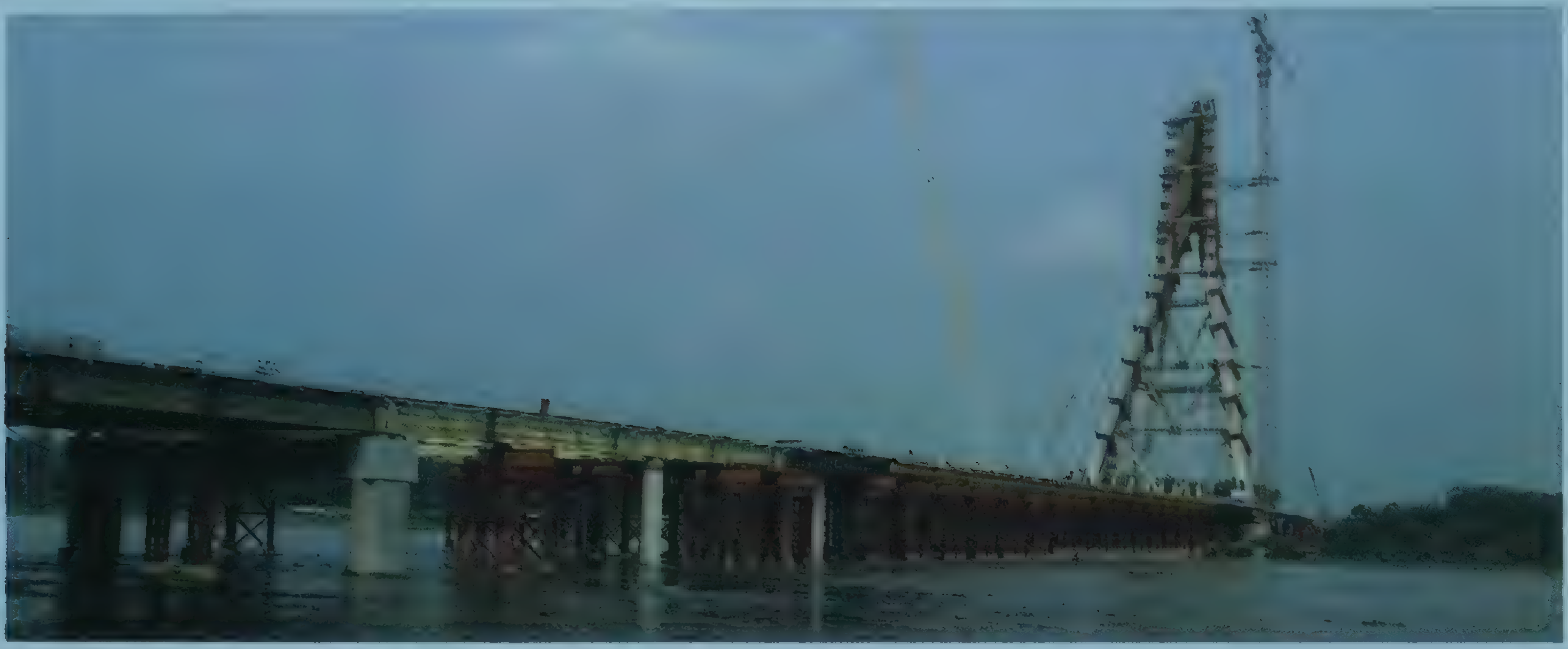


Fig. 5.1: Deck Erection by Staging Method (Source: JV of Gammon-Cidade - Tensaccai)

The erection of cable stay deck by staging method is realized in following 4 steps:

- ✓ Erection of the deck on the permanent piers and the temporary supports. In this stage the erection is actually a normal girder erection and any of the procedures used for the construction of girder bridges can consequently be applied.
- ✓ Erection of the pylons from the completed deck, serving as a platform.
- ✓ Installation of the stay cables. In this stage the cables only need to be tensioned moderately as the final cable tension will be induced in the last stage.
- ✓ After installation of all stay cables, the temporary supports are removed and the load transferred to the cable system. During this process the deck will deflect downwards and it is therefore necessary to initially erect the deck in an elevated position to arrive at the desired final geometry when the total dead load of the deck has been transferred to the stay cables.

With the deck erected on temporary supports and subsequently lowered, a permanent bending will be induced in the deck sections. With slender decks the bending moments might well be within acceptable limits, but with a stiffer deck the procedure might have to be modified (to eliminate this permanent bending). In that case the deck must initially be erected with its final elevation on the temporary supports and then be jacked up before the stay cables are attached and partly stressed.

The erection procedure illustrated in **Fig. 5.1** offers the advantage that the deck can be erected continuously from one end to the other and thereby allows transportation of men, equipment, and material on the completed part of the deck. Also, the procedure leads to an efficient control of the geometry and the cable tension.

The disadvantage of the procedure is related to the temporary supports that are required. In many cases a clearance profile will have to be respected also during the construction period so that temporary supports cannot be allowed. Even if the clearance requirements are not prohibitive, the cost of erecting the temporary supports, e.g. on large water depths in the main span, might be of such a magnitude that the procedure will not be feasible.

5.2.2 *Push-launching method*

The push launching method is sporadically used for girder bridges, and, as such the concept is not entirely new for cable-stayed bridges. In this method of erection, a large area behind the abutment is utilized as the assembly shop. The length of the area required behind the abutment must be larger than the side span of cable stayed bridge. The cable stays can be laid out on the deck next to the pylon and erected along with the saddles to the top of the pylon by a crane. Then the assembled span along with the pylon and cables is pushed out, cantilevering from the abutment to the predetermined length. Then the prefabricated modules are added behind and the span is further pushed out till the front part of the span rests on permanent or temporary piers. The operation continues.

The erection of cable stay deck by push launching method is realized as shown in the **Fig. 5.2**. In this respect the following is to be noted:

- ✓ In the final position the reaction load of the towers is borne by the permanent piers VIII and XI. However, during the push-out operation the tower reaction must be resisted by a lateral-beam diaphragm which in turn transmits the load to the longitudinal box girders.
- ✓ For this reason the cable stays are only partially tensioned. The jacking mechanism at the saddle is used to compensate for the cantilever deflection of the leading edge of the pushed-out section.
- ✓ When the leading edge of the bridge reaches pier VIII, the bearing is elevated to the designed height by jacks. As a result of this action the bearing pressure at pier VII is relieved.
- ✓ As the structure is pushed out farther, the bearing pressure at pier VIII will increase. Once the bearing pressure reaches its allowable limit when the leading edge extends by the designed length beyond pier IX, the bearing at pier VIII is lowered to its original position.
- ✓ This procedure is then repeated until the structure is in its final position.

The erection of the Millau Viaduct in France, **Fig. 5.3** is one of the most spectacular bridge erections ever performed by push launching method. It was especially challenging as the pre-fabricated segments could not be brought to the site and lifted vertically into place as generally occurs in cable supported bridges across waterways. Instead the super structure was push-launched from both abutments to a closing point above the Tarn River 270 m below the deck.

To give the necessary support during launching the permanent piers had to be supplemented by temporary piers at the centre of each span. The closing point between the two launched deck sections was positioned at the centre of the span above the Tarn River, where the deck is highest above the ground. Thus a temporary pier could be avoided where it would have been the longest.

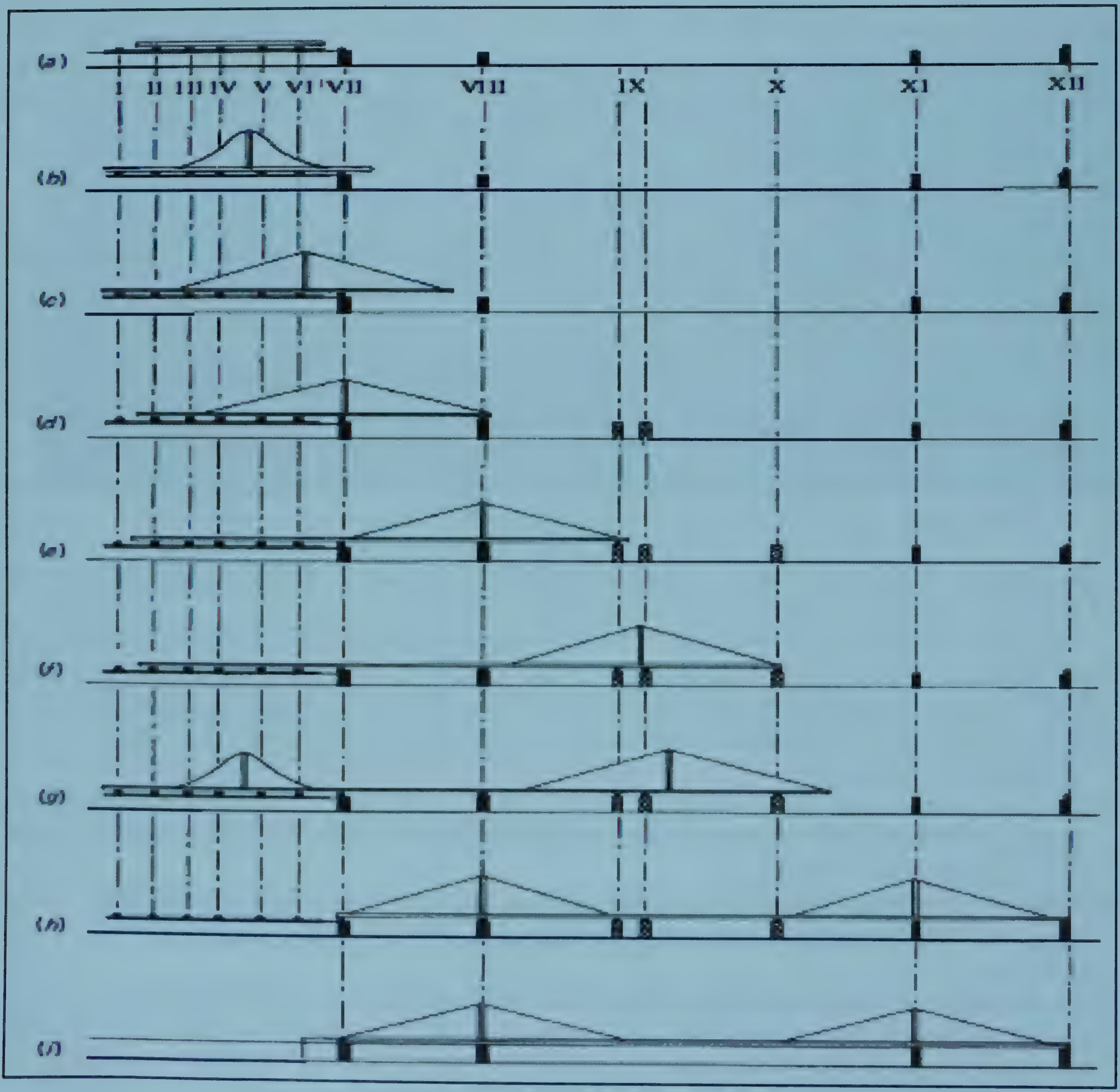


Fig. 5.2 Erection Stages for Push Launching Method [3]

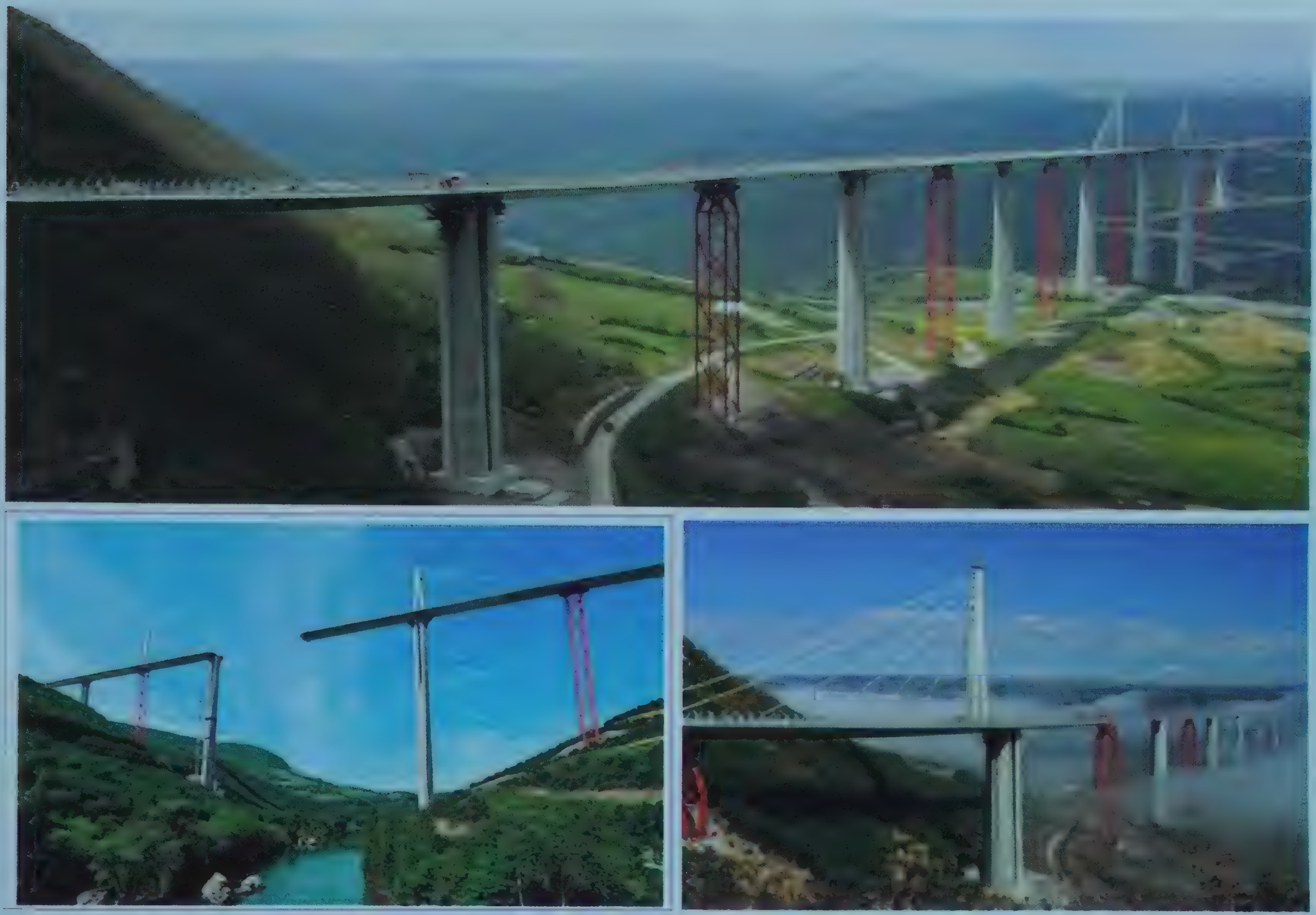


Fig. 5.3 The Millau Viaduct during the launching. The deck is partly supported by the temporary supports until the remaining pylons and cable systems are added

5.2.3 *Cantilevering method of erection*

Where it is necessary to avoid temporary supports in the spans, cantilever method of construction is adopted for erection. There are two types of cantilever methods of construction:

- Balanced cantilever method of construction;
- Free cantilever method of construction.

For using this technique, construction plan shall be made taking into account the conditions of construction site, economic factors and scale of construction. Especially, it is necessary to consider cable arrangement and the sequence of prestressing. If the deck is of cast in situ construction, the concreting is done in form traveller in 3 to 5 m long segments along the axis of the bridge. In some cases, the use of special form traveler which are modified in order to avoid intersection of main frame and anchorage of stay cable, or equipped with facilities for moving back may be required. Since the cross section of girder of cable-stayed bridges is usually constant, the same formwork may be repeatedly used. Also, because the height of the girder is comparatively low and web is often slanted, materials and structure of formwork should be appropriately chosen depending on its objective. Inside the box girder, on the upper surface of the slanted web, wooden formwork attached fiber sheet can be used to prevent the formation of bubbles.

In case of cantilever erection of cable-stayed bridges, anchorage blister and cross beam which are arranged at the anchorage of stay cable are cast with the girder because of the convenience of prestressing of stay cable. There may be a situation when the formwork is needed to be assembled and disassembled every time the wagon moves. Therefore, it is necessary that structure of formwork for inside of girder is planned so that proper workability is maintained.

For precast segmental construction or steel composite deck construction, precast segments or steel girders in modules are prefabricated at the fabrication yard, transported to the location and connected to the adjoining segments already positioned in case of precast segments or steel module. Depending upon the available logistics, the precast segments or steel modules are handled by segment launcher or barge mounted cranes and held in position before being connected to adjoining segment already positioned and prestressed, or, connected by bolting to adjoining steel module already positioned.

Comparing with other girder and extradosed bridges, cable-stayed bridges have larger displacement and vibration during casting of concrete, larger vertical and longitudinal displacement by change of temperature (temperature gradient and uniform temperature) and horizontal displacement due to wind. Therefore, when constructing the closure part, if the displacement and vibration by casting concrete, change of temperature and wind are large, the countermeasures for these problems shall be implemented. The false work mechanism which can control and suppress these displacement and vibrations shall be used if necessary.

Also, consideration to cast the concrete or place the precast/steel segments during the time when change of temperature is relatively small in a day is required if the displacement by change of temperature is large.

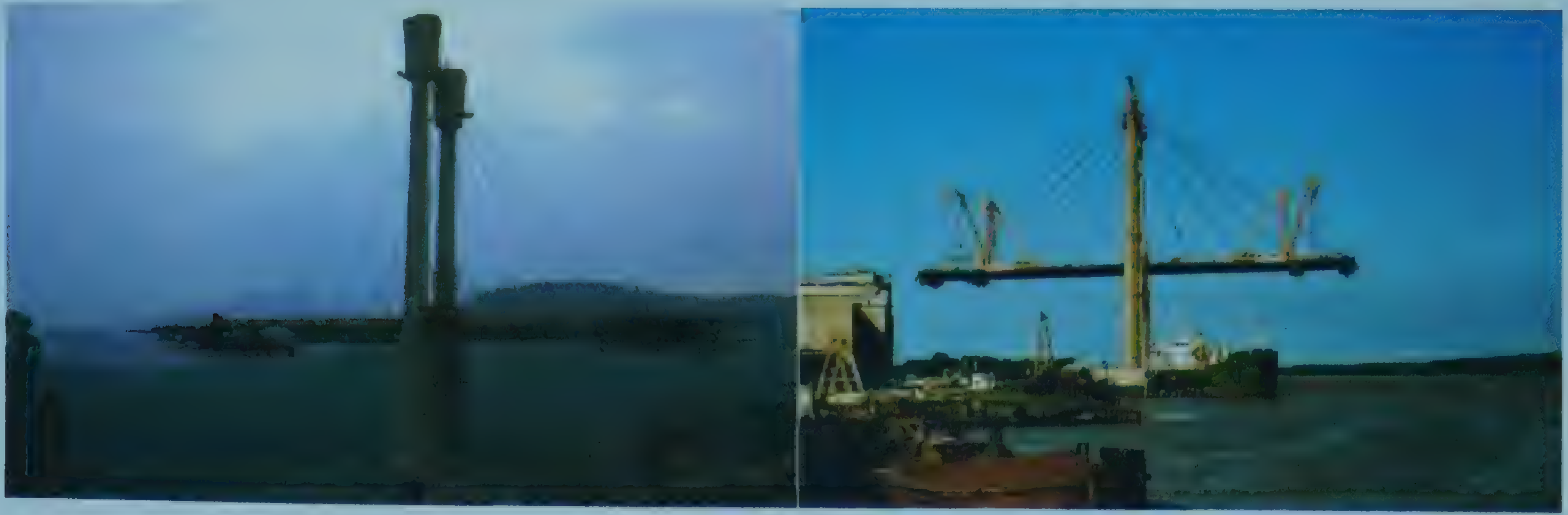


Fig. 5.4 Balanced Cantilever Method of Erection (Source: Left: Gammon India ltd, Right: [1])

The erection of cable stayed deck by balanced cantilever method is realized in the following steps as shown in the **Fig. 5.4**:

- ✓ The pylons (and the deck units above the main piers) are erected and fixed to the piers.
- ✓ A balanced cantilever on either side of the pier is initiated using segment lifters/derrick cranes operating on the deck to lift deck segments transported to the site on barges/trailers. The large capacity cranes can also be used from the ground/river if the ground accessible or crane can be barge mounted.
- ✓ As the cantilevers grow, the stay cables are installed and tensioned initially to relieve the bending moments in the deck. Often the cantilever process is completed for one bridge half, before moving the cranes to the other half.
- ✓ The deck is joined at midspan and additional loading from wearing surface, railings, etc., is applied.

With this procedure it is essential to have a very efficient fixity of the superstructure to the pylon piers throughout the Construction period, as the entire stability depends on this fixity until the end pier is reached. Also, the lateral flexural stiffness of the deck must be sufficient to ensure the stability of the cantilever arm that will eventually reach a length corresponding to half the main span length. Thus, the procedure is especially advantageous in bridges with a reasonably large width-to-span ratio of the deck.

The balanced cantilever method of erection is most efficient if the distance between the cable anchor points is chosen so that the deck can be cantilevered from one cable anchor point to the next without requiring a temporary support (e.g. by temporary cables). This condition is often fulfilled in bridges with multi-cable systems.

In free cantilever method of erection the side spans are erected on temporary supports and the main span by a free cantilever, as shown in **Fig. 5.5**.

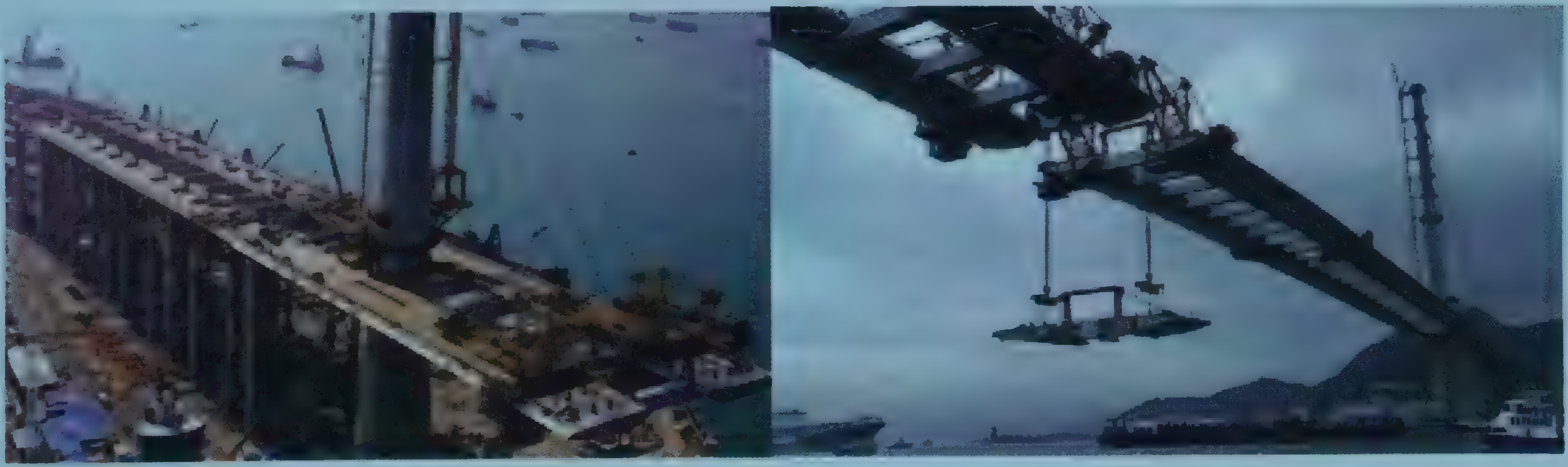


Fig.5.5 Free cantilever Method of Erection [1]

The erection of cable stay deck by free cantilever method is realized in the following steps:

- ✓ Erection of the deck in the side spans using temporary supports, followed by the erection of the pylons when the main piers are reached.
- ✓ One-sided free cantilever of the main span deck with installation of both the main span and the corresponding side span stay cables as the relevant anchor points in the main span are reached.
- ✓ Completion of one half of the bridge, followed by the cantilevering of the other half.
- ✓ Closing of the bridge at the main span centre.

With this erection procedure, the application of temporary supports is limited to the side spans where clearance requirements and water depths seldom will exclude such supports. At the same time the cantilever into the main span can start from a very stable position with the side span deck supported on both the main pier and the end pier. A temporary moment-rigid fixation to the main pier is therefore not required.

5.2.4 *Temporary cable system during erection*

- Temporary erection stays can be used in cable stayed bridges with few stay cables, resulting in considerable distance between the cable supported points of the deck. In these bridges overstressing might occur either in the deck section or in the last erected stay cable from which free cantilevering is being done. In such a case a temporary erection stay, as shown in **Fig. 5.6** (Left), might be installed to reduce both the bending moments in the deck and the tension in the permanent stay cable.

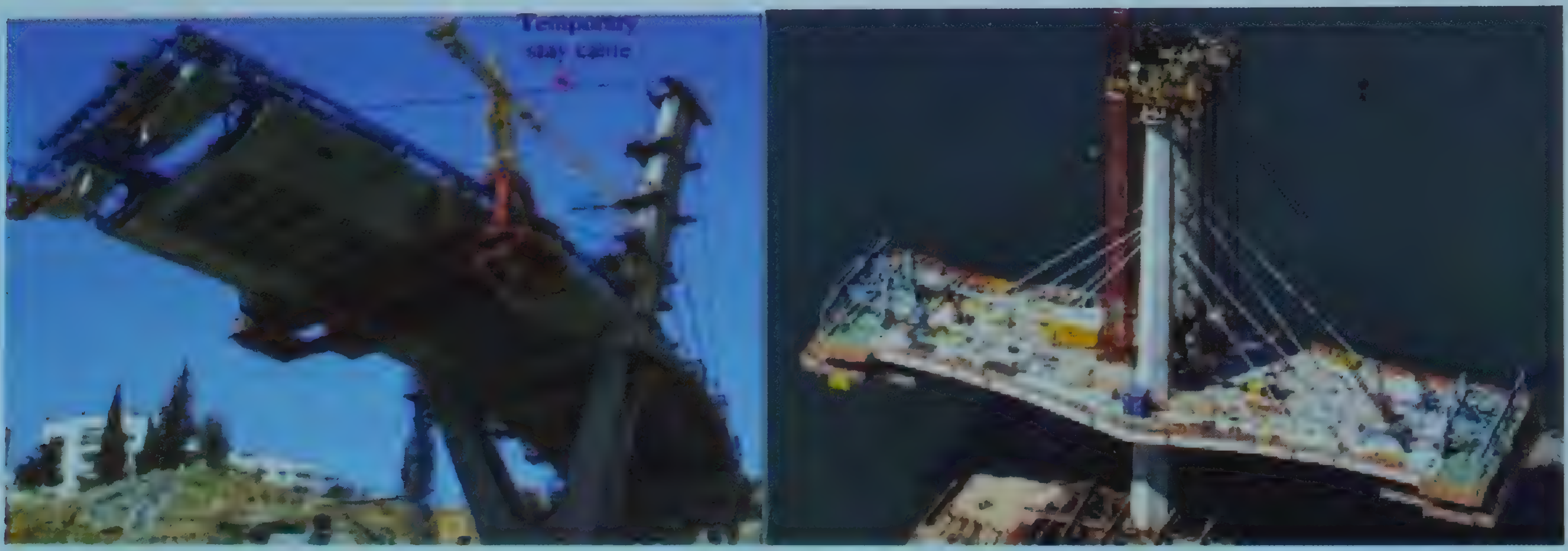


Fig.5.6 Temporary stays in the pylon (Source: Left: L&T, Right: [1])

As an alternative arrangement, the cantilevered part of the deck may be supported by a secondary cable system comprising a temporary pylon, with a set of temporary erection stays, as illustrated in **Fig. 5.6** With this arrangement, the hogging moment in the deck is reduced, whereas the tension in the permanent stay cables remains practically unchanged. Thus, this arrangement is only applicable if the problem of overstressing is confined to the deck.

However, it may be noted that temporary measures in the form of supports, stays, and pylons used only during erection are costly as they have to be fabricated, erected, and demolished before the construction is completed. Therefore, it becomes imperative that various aspects of erection are taken into consideration in the very early design stages so that the main layout of the structure can be chosen to minimize the amount of temporary structural members.

5.2.5 *Construction Methods based on type of deck / material used for deck*

Selection of deck type in Cable-stayed bridges is key parameter in terms of project costs and construction period. Based on the deck system, feasible erection methods shall be studied and decided upon, taking into account the maximum weight and length of the element that can be lifted or cast, the material as well as construction cost in terms of equipment, facility and time, such as to lead to the most economical system.

5.2.5.1 Concrete Deck

If concrete deck is adopted in a cable stayed bridge, most feasible construction method of deck is either cast-in-place deck or precast segment deck erection. It has to be chosen based on overall length of bridge, cost of precasting facility and accessibility of transportation etc.

Precast Elements

The advantage of precast elements is their fabrication under controlled conditions, independent of adverse weather. Segment weight and length shall be decided considering the lifting capacity of derrick crane and cycle time of erection. Longer segments require heavier lifting capacity in general and may require more temporary tensioning such as Post tensioning, or, ancillary stay cables.

The precast elements can be floated to site on barges or transported through truck, and, lifted by derricks located on each tip of the already erected beam, cantilevered to both sides (of the tower **Fig. 5.7 (a) & (b).** The elements are placed in such a way that they are balanced on either side about the tower. Each derrick is tied back to erected deck temporarily.

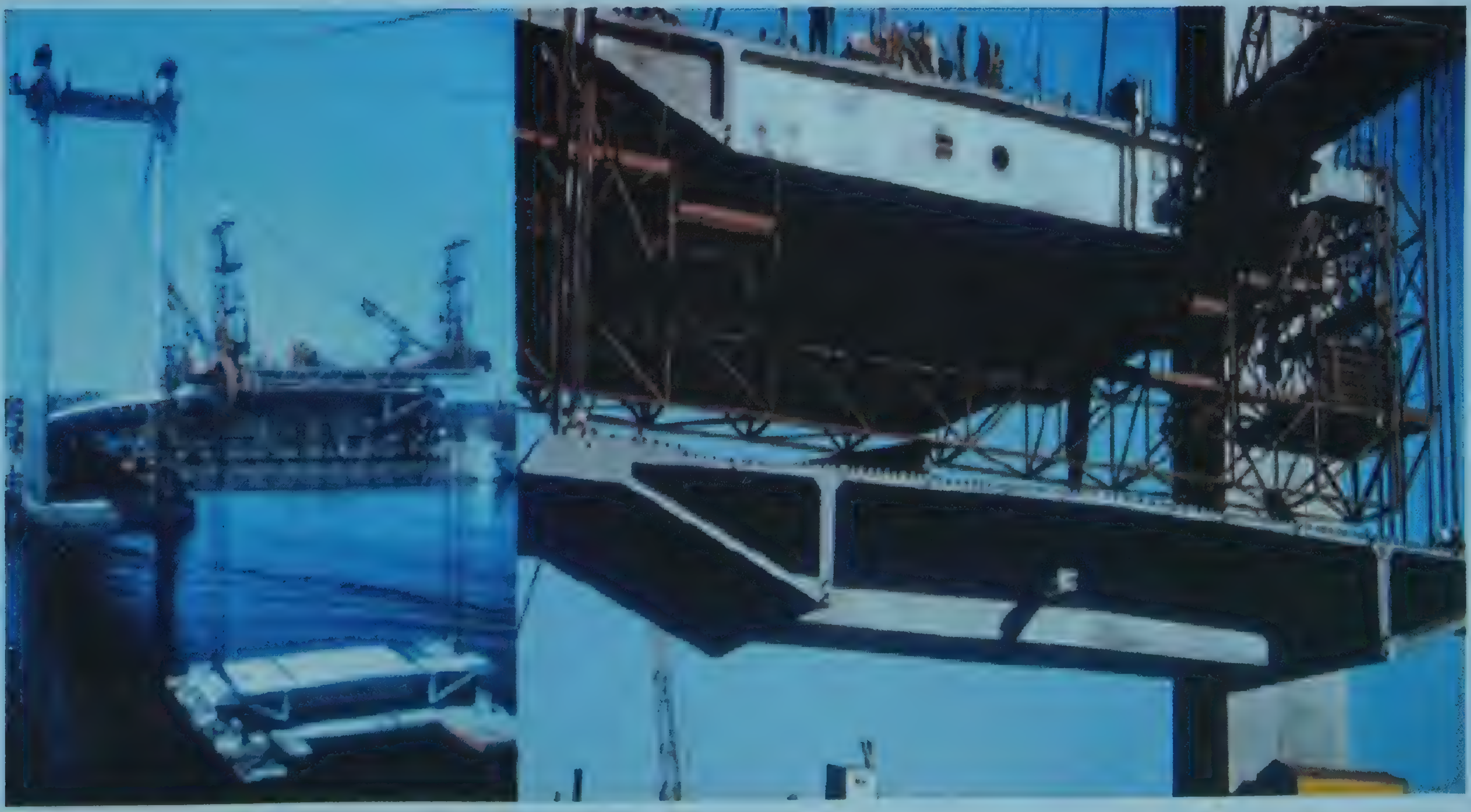


Fig. 5.7 (a) Lifting of Precast Segment (Pasco Kennewick Bridge, Washington, [1])

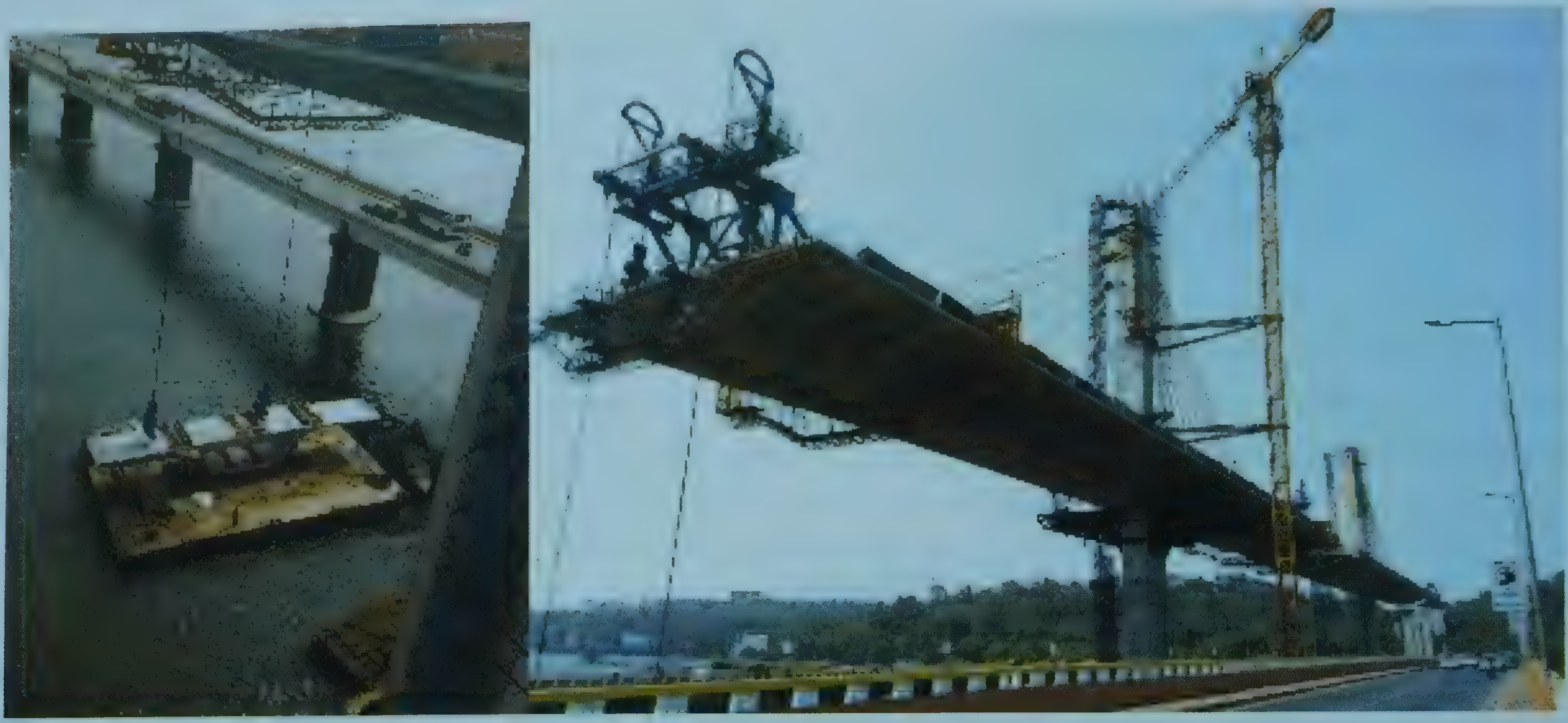


Fig. 5.7 (b) Lifting of Precast Segment Cable Stayed Bridge across River Mandovi, Goa, (Source: L&T)

Cast-in-place (CIP) Concrete

Cast-in-place deck is often cast by application of a form traveller attached to the completed part of the deck, **Fig. 5.8.** With the modest depth of the edge girders generally found in these bridges, the moment capacity of the deck is quite limited, so it will be necessary to cast the deck in relatively short segments with a length of typically 4 m - 6 m and the full width of the deck. The form traveller has to be designed so that it can be moved forward and cantilevered after the newly cast concrete segment has hardened and the permanent stays are installed and tensioned. With a reasonably short segment length in each casting, it will be possible to avoid temporary cables and it may not be necessary to strengthen the edge girders for the loading that comes during construction.

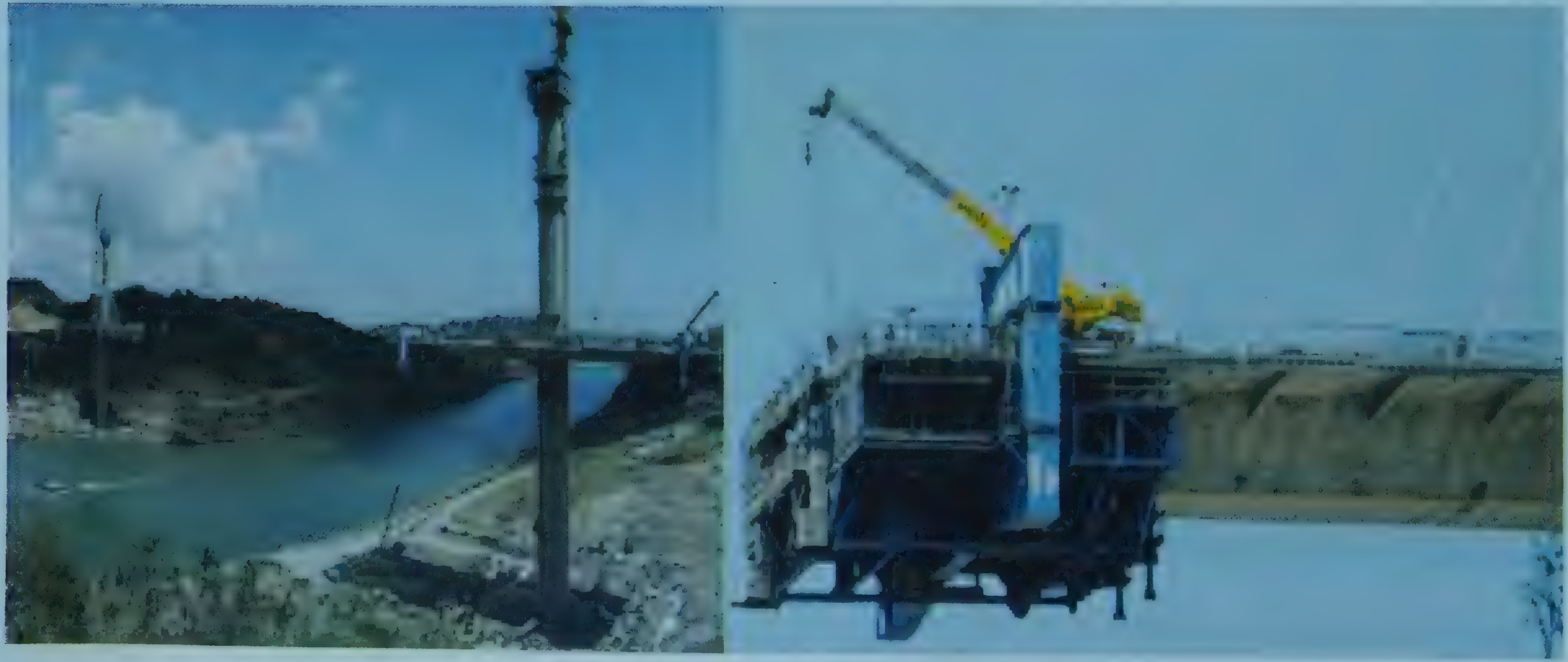


Fig. 5.8 Balanced Cantilever Erection by form Traveler [1]

5.2.5.2 *Steel Deck*

The usual method of building the main span steel deck is free cantilevering which is one of the main advantages of cable-stayed bridges. The deck sections with a length of 10 to 20 m, equal to the cable distance, are completely preassembled and transported to site on water, if possible, or on the already installed deck. The lifting takes place either by derrick from the already erected deck or by floating crane, **Fig 5.9 (a)**. Large block erection of orthotropic steel deck (maximum length of 112.7 m and 2302 t) using heavy lifting capacity of floating crane was carried out for the erection of back span in Incheon Bridge followed by the small block erection using the derrick crane in main span, **Fig. 5.9 (b)**

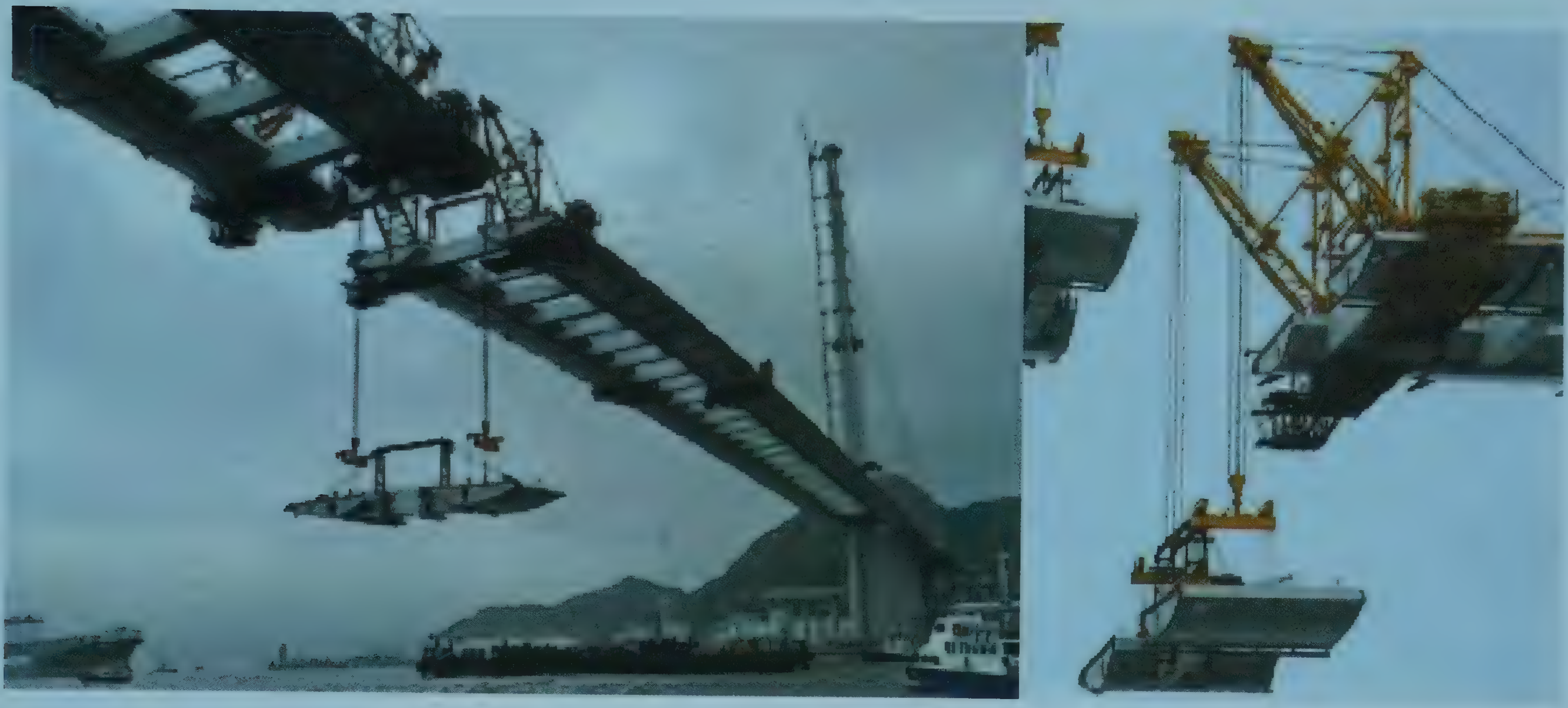


Fig. 5.9 (a) Lifting of Steel Deck [1]

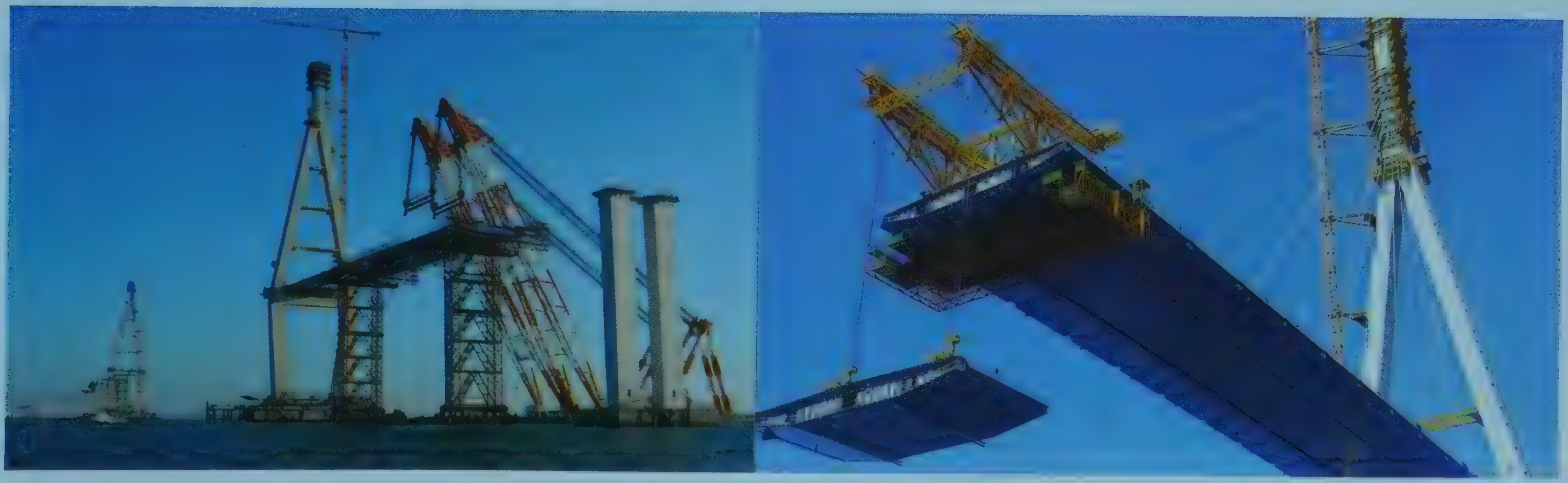


Fig. 5.9 (b) Large Block Erection in Back Span (left) and Small Block Erection in Main Span (right) (Source: Incheon Bridge Company)

5.2.5.3 *Steel Composite Deck*

Composite deck is built from a steel grillage (**Fig. 5.10 (a) & 4.24**) on which rests a concrete slab, either CIP or precast. **Fig. 5.10 (b)** shows the Atal Bridge in which the side spans as well as deck is made in CIP concrete.

During free cantilevering, the grid sections are lifted by derrick and welded to the already erected beam between the cable points. Then the concrete slab is installed. The steel beam should not run ahead of the concrete slab because in that case the action forces are very difficult to determine due to shrinkage and creep.

Precast slab panels are advantageous for rapid erection and reduced shrinkage and creep values due to their advanced age at installation. One disadvantage is their joints, which are mostly in compression in both the directions under permanent loads.

More time is required for false work of the slab and placement of reinforcement in case of cast-in-place slab.

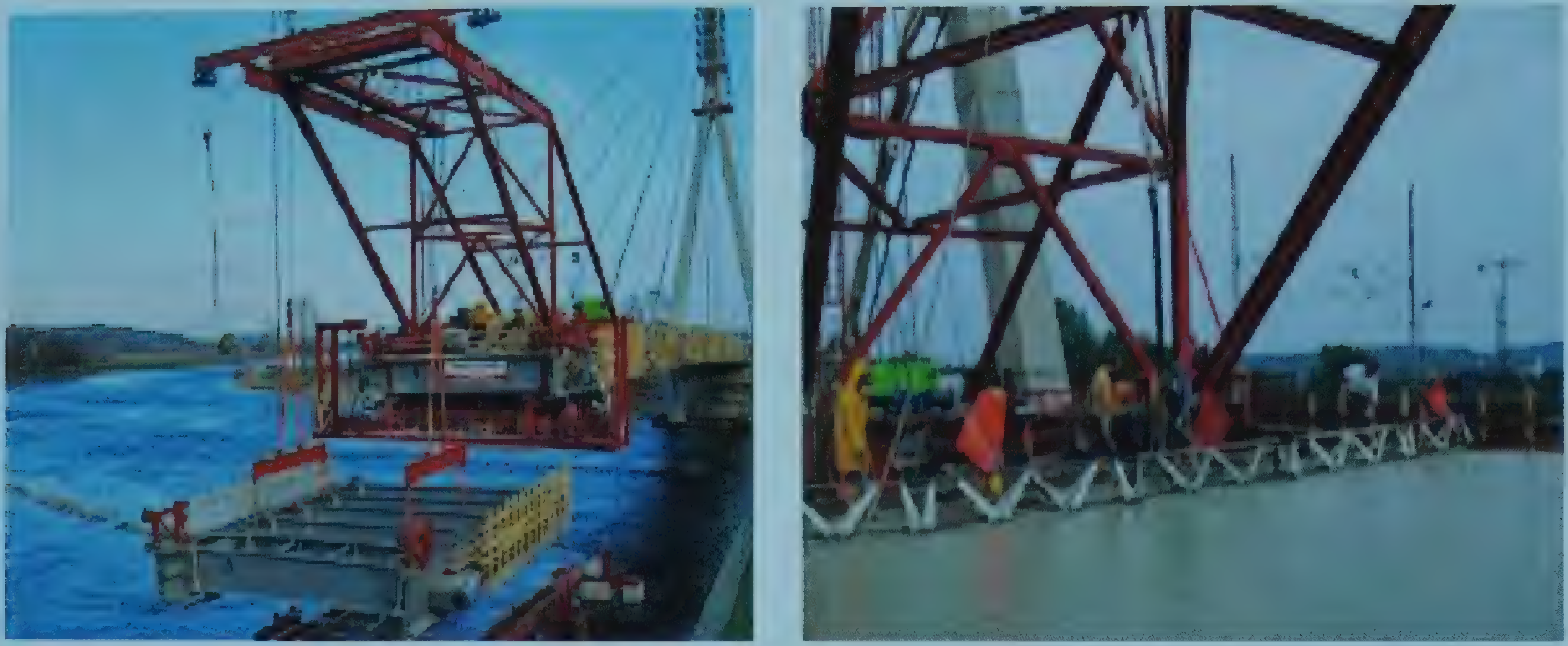


Fig. 5.10 (a) Lifting of Steel Grillage and Cast-in-place Concrete [1]

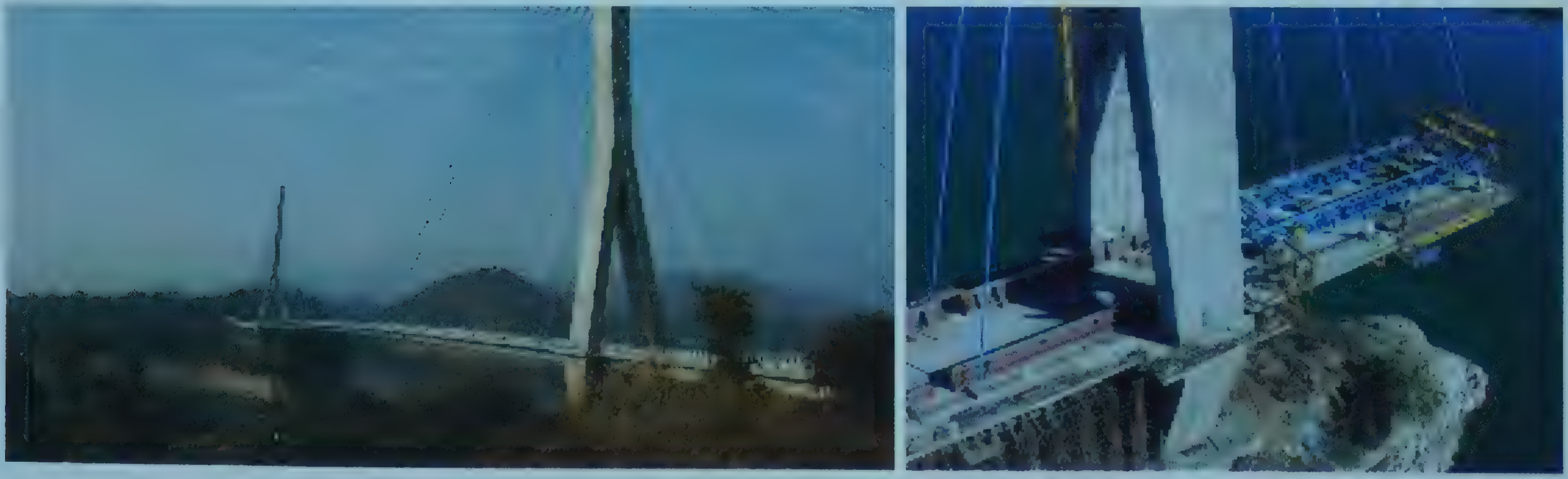


Fig. 5.10 (b) Steel Composite with Cast-in-Place Concrete (Source: Basholi Cable Stayed Bridge, J&K, S P Singla)

5.3 Construction of Pylon

The methods used for the construction of pylons are generally the same as for other tall, slender structures. The pylons could be in concrete or steel. Traditionally, pylons are designed with a pronounced slenderness in the longitudinal direction of the bridge to allow the pylon to follow the longitudinal displacements of the cable system. However, during construction of the pylons, the cable system is not in place so the pylons must possess the necessary resistance against horizontal wind load by themselves. Often the most critical phase in the construction of the pylon occurs in the period from when the pylon is at full height until the first stay cables or the catwalks are erected. In this period wind-induced oscillations of the pylon in the longitudinal direction of the bridge might be experienced. These are generally caused by the slenderness of the pylon in this direction, combined with the large area of the cross beam at the top exposed to wind and the climbing crane.

The problems related to wind-induced oscillations of the free-standing pylons will generally be more pronounced for steel pylons than for concrete pylons due to the steel structure's smaller mass, larger flexibility, and lower damping. In a number of cases it has therefore been necessary to take special measures to achieve aerodynamic stability in the critical construction phases. In Japan, larger pylons often have had tuned-mass dampers installed temporarily once a certain height has been reached and until a supplementary support is provided by the cable system. For major cable stay bridges with very tall pylons it is recommended to perform wind tunnel tests to check the stability of the freestanding pylon and to determine in which phases it will be necessary to take precautionary measures.

As explained earlier, the pylon must be flexible to follow the displacement of the cable system, and as the displacement of the pylon top increases significantly with the side span length, the problems of fulfilling the requirements of stability during construction and flexibility in the final stage will be more severe in bridges with large side-to-main span ratios. If both requirements cannot be fulfilled, a temporary support of the pylon in the longitudinal direction may become a necessity.

5.3.1 *Concrete pylons*

For formwork, large wooden panel system is usually used. Wooden formwork is also generally used for the anchorages of stay cable. For saddle construction, since it has a complicated shape, pre-fabricated steel formworks can be used. To save on labor and to maintain sufficient durability, precast concrete panel may also be used as formwork which is left in place after casting.

Since sectional force in a pylon is very large, densely arranged reinforcement is inevitable, because of the pylon being a slender structure. he accuracy of reinforcement processing, assembling and connecting has large effect on the workability, structural strength and durability. Therefore, the reinforcement must be processed and arranged exactly in accordance with design documents.

If a steel frame (**Fig. 5.15**) is used, it secures the construction accuracy of the tower, and also helps in fixing the reinforcing bars and formwork. In case the saddle is used for fixing stay cables, improvement of safety and workability in the site work is required such that the steel frame can be used as a structure supporting the saddle frame. The steel frame is fabricated in blocks. The blocks are then temporarily assembled to check the fabrication accuracy such that the verticality of the connection device and anchor material of the stay cable or saddle is ensured. Then blocks are transported to construction site and assembled. It is essential to precisely arrange and set the pivot block at the construction site.

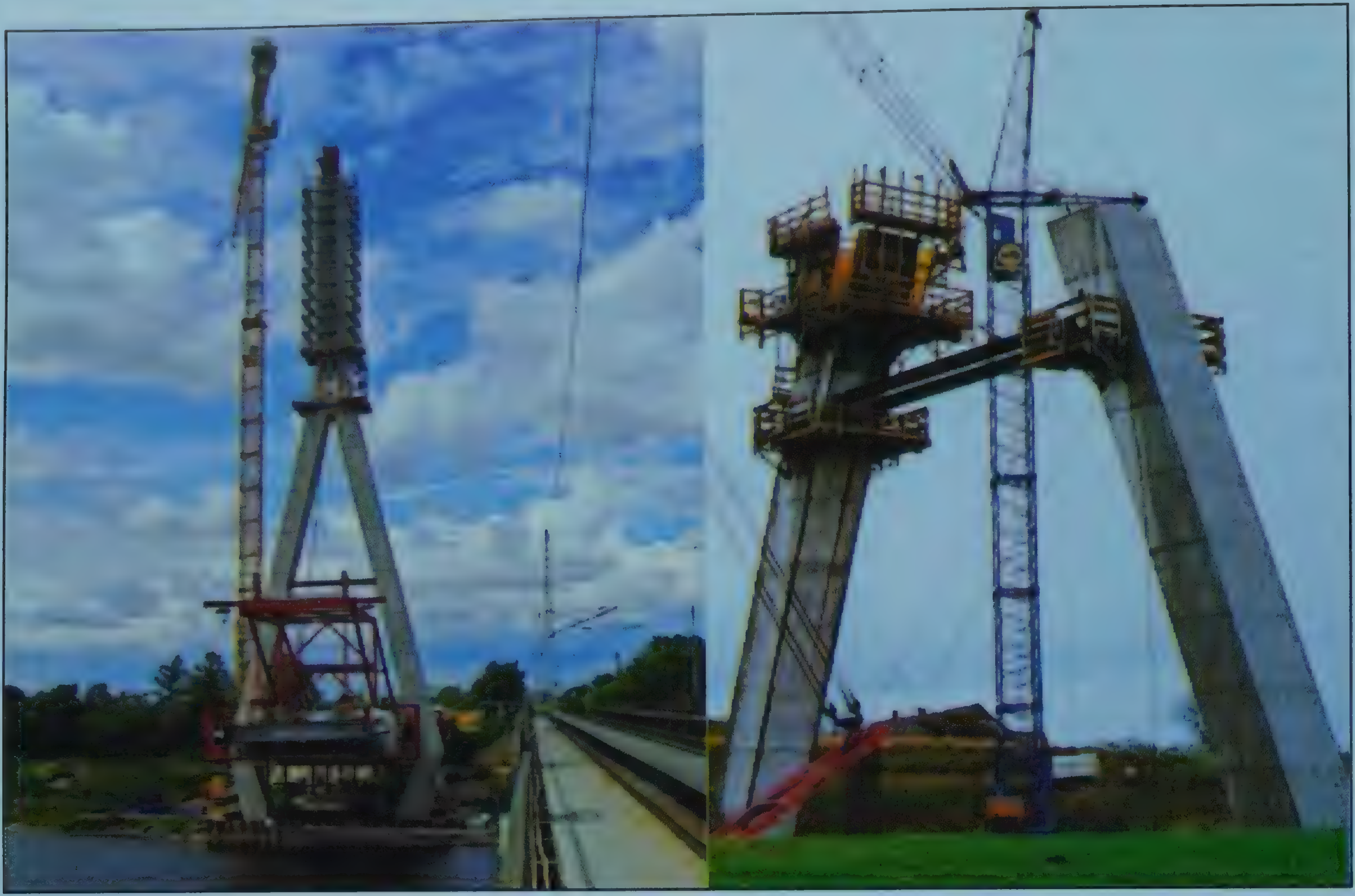


Fig 5.11 Concrete Pylon During Construction showing, Tower Crane, Hoists, Climbing Scaffolding and Falsework for Cross Beam [1]

The full scaffolding assembled around the pylon frame is the most common method used for the construction of pylon. It has a merit that it can be applicable to the complicated pylon shapes with high versatility though, assembling and disassembling are carried out at a high location and therefore maintaining the safety is a main concern. On the other hand, in case of the climbing scaffolding, **Fig. 5.11,** which is mechanically moved for hoisting, excess facility such as tower crane, elevator become necessary, but this is much better in terms of safety. The climbing scaffolding system could be either jump formed system or slip formed. Jump forming is preferred over slip forming as the former provides more flexibility for insertion of various add-ons during construction of pylons.

For constructing horizontal cross beam for pylon of cable-stayed bridges, generally the falsework is placed on steel beam supported by brackets which are fixed at tower surface. If the pylon is slanted, bending moment is produced; therefore, temporary strut or reinforcement by HTS may be required to resist the same. Also, it is necessary to plan the structure and support method of temporary strut with consideration of dismantling them.

Hoist facilities used in construction are determined by terrain conditions, shape of tower and weight of load to lift. However, tower crane is commonly used when the height of pylon or pier is high. When the height of tower crane becomes high, stay support of crane fixed to the pylon structure is needed. In this case, it is necessary to examine for the local stress generated in the pylon concrete due to the reaction of stay.

5.3.2 *Steel pylons*

For smaller steel pylons of moderate size cable stayed bridges the erection might be carried out by mobile cranes with high booms or floating cranes, **Fig.5.12**. Erection by land or sea-based cranes may also be used for at least the lower parts of high pylons. With the evolution in size of floating cranes experienced since the 1970's it has become possible to erect the entire pylon in one or two pieces.

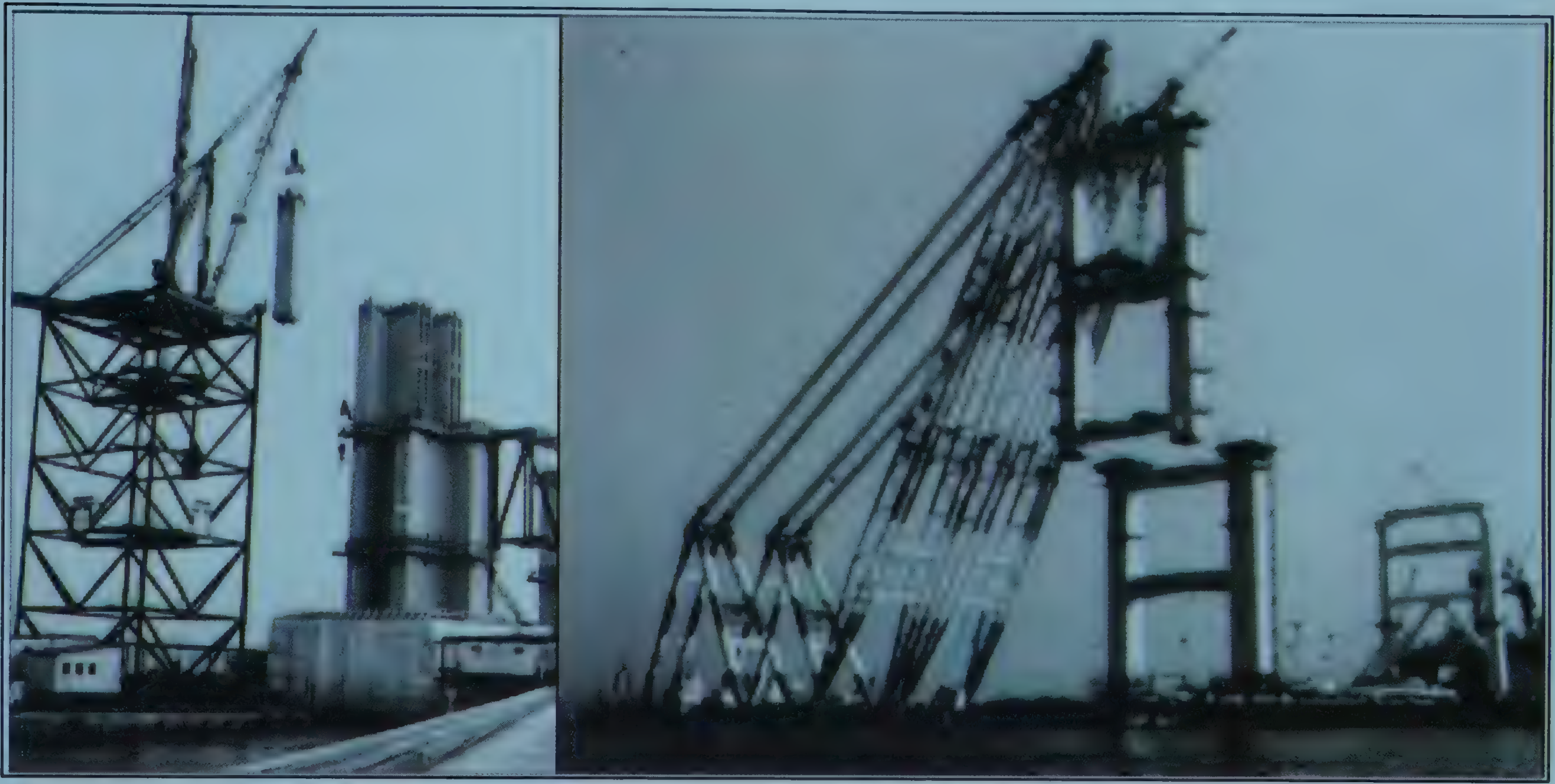


Fig.5.12 Erection by floating derrick cranes [3]

For very high pylons it is necessary to erect at least the upper part by the traditional procedure where a climbing crane follows the pylon as it grows. The crane applied to hoist the prefabricated pylon units into place generally consists of a derrick crane on a latticed cross beam attached to both pylon legs, as illustrated by the erection of the pylons for 2nd Hooghly bridge (Vidyasagar setu) at Kolkata, **Fig. 5.13 (Left)**.

With the advent of very high capacity cranes, the very tall pylons may be erected by crane using prefabricated segmental construction technology. In signature bridge at Delhi, 154 m tall slanting pylon is erected segmentally by 1250 t capacity crane as illustrated in the **Fig. 5.13 (Right).** A tower crane is also connected to pylons which is attached with passenger hoist for transportation of materials, equipment for cable stressing and installation, and, crews.

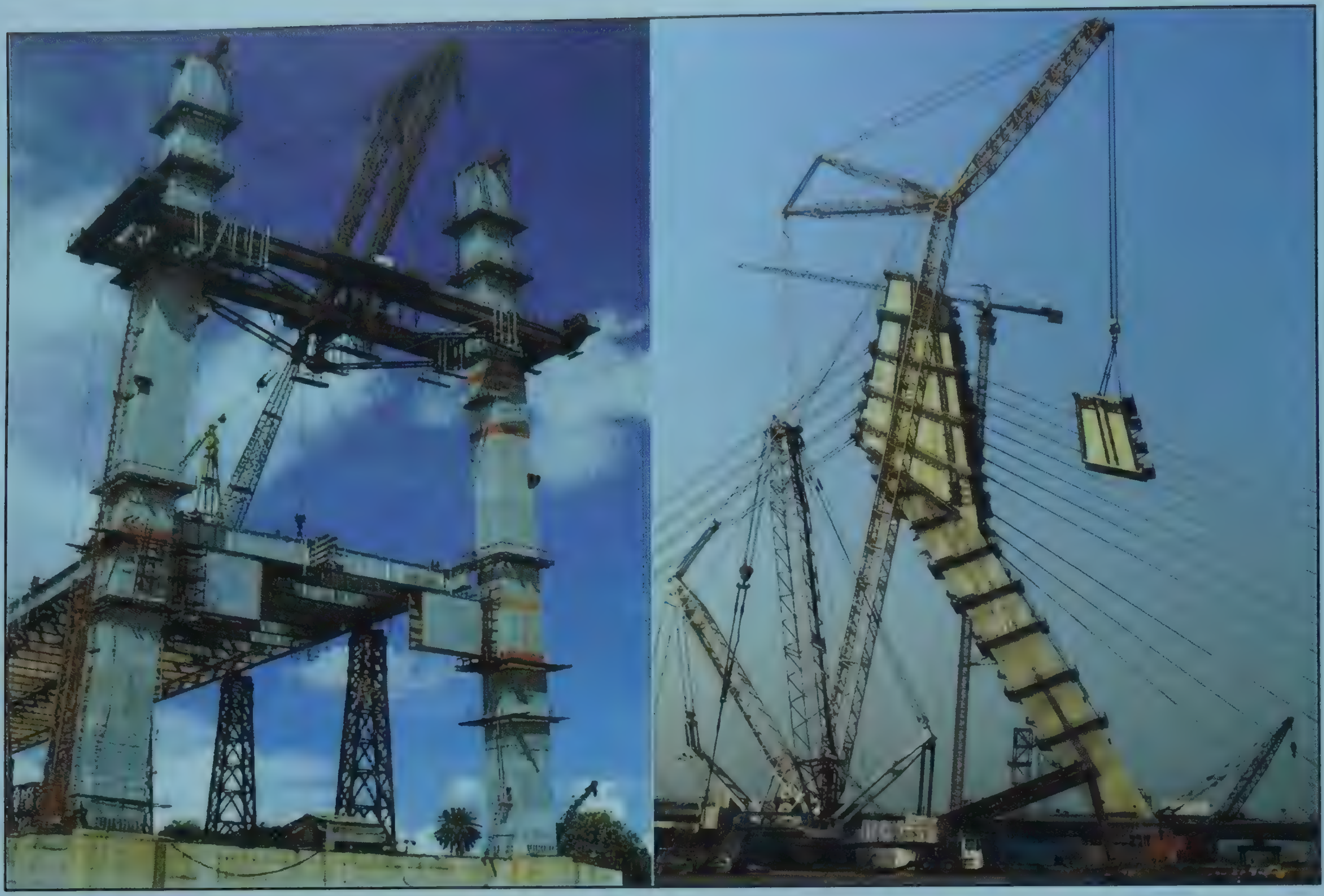


Fig. 5.13 Prefabricated segmental erection of steel pylons (Source: Left: Gammon India Ltd, Right: JV of Gammon - Cidade - Tensaccai)

5.4 Construction of Saddle and Anchorage of Stay Cables

The installation position and inclination alignments of stay cable change through the duration of construction due to the changes of prestressing force and deflection of pylon and deformation of other members. However, basically, the positions during service of bridges shall be adjusted precisely in order to conform to design calculations. Therefore, in setting the saddle and anchorage of stay cables, the effects of sag of stay cable, camber when the main girder is constructed, concrete creep and shrinkage shall be considered to ensure that the saddle and anchorage will be in precise position during service of bridge.

The anchorage assembly at deck or beam level has to be properly designed depending upon the type of the deck, whether the deck is of concrete, steel or composite. The typical assembly in concrete and steel or composite deck is given in **Fig 5.14**. The design of cable anchorages should be such that if required, the assembly should enable the replacement of cables with out complexities. Generally, steel casing pipe is embedded inside the concrete member and the stay pipe including the cable is placed in the casing pipe. Where the deck is steel or composite, the anchorage assembly is welded to the girder while the bottom plate of the anchorage assembly is fitted into the grove on top of the girder to take care of the horizontal forces arising out of the inclination of the stay cable. It is imperative to carry out the special water proof and anti corrosive treatment surrounding the area where anchorage assembly meets the deck area.

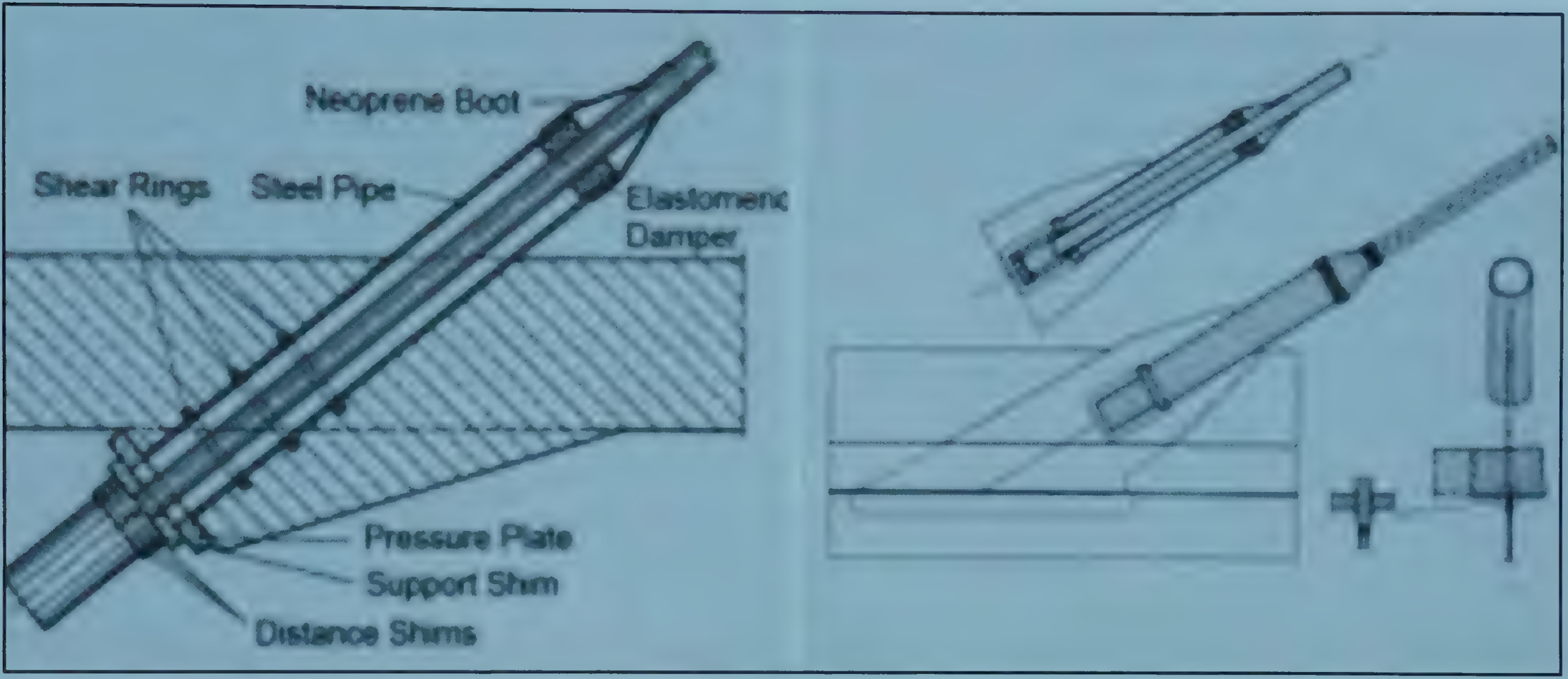

Neoprene Boot
Shear Rings
Steel Pipe
Elastomeric Damper
Pressure Plate
Support Shim
Distance Shims

Fig. 5.14 Anchorage Assembly at Deck level for Concrete (Left) and Composite (right) girders [1]

Usually the anchorage device is fixed with the embedded steel frame, **Fig. 5.15** inside the concrete of pylon. If steel frame is used for installation of the anchorage, there are the cases of using adjustment tools, which allows micro adjustment both of up-down and left-right for cable anchorage. In order to secure the accuracy of saddle installation position, it is desirable to unitize the unit structure, as shown in **Fig. 5.15 (Right)**

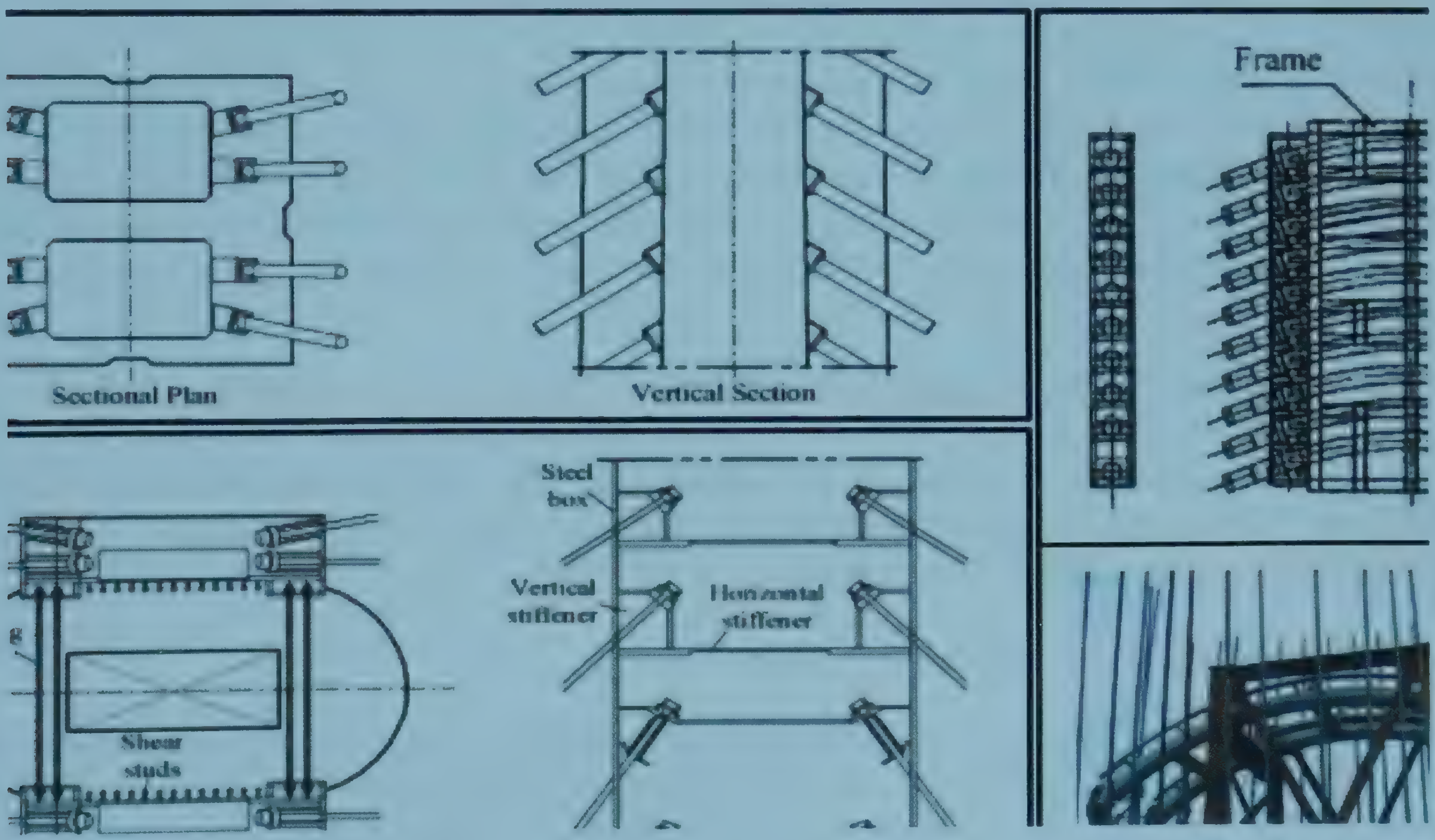

Sectional Plan
Vertical Section
Frame
Steel box
Vertical stiffener
Horizontal stiffener
Shear studs

Fig. 5.15 Left : Steel Frame Inside and Outside Concrete Pylon, Right: Steel Frame for Precise Positioning of Saddles inside Concrete Pylon [2]

Special attention should be paid to ensure that no local bending of the cables occurs due to use of the saddle, which is composed of the HDPE pipe, and steel pipe, which are bent to the required shape. Therefore, it is necessary to make the pipe edge of the saddle free from local bending. In cases that the lengths of the cables are extremely long, it is also necessary to investigate a structure, which can absorb unexpected effects such as construction error and sag effect. Also, it is necessary to consider these matters with regards to not only saddle but also anchorage structures for stay cable.

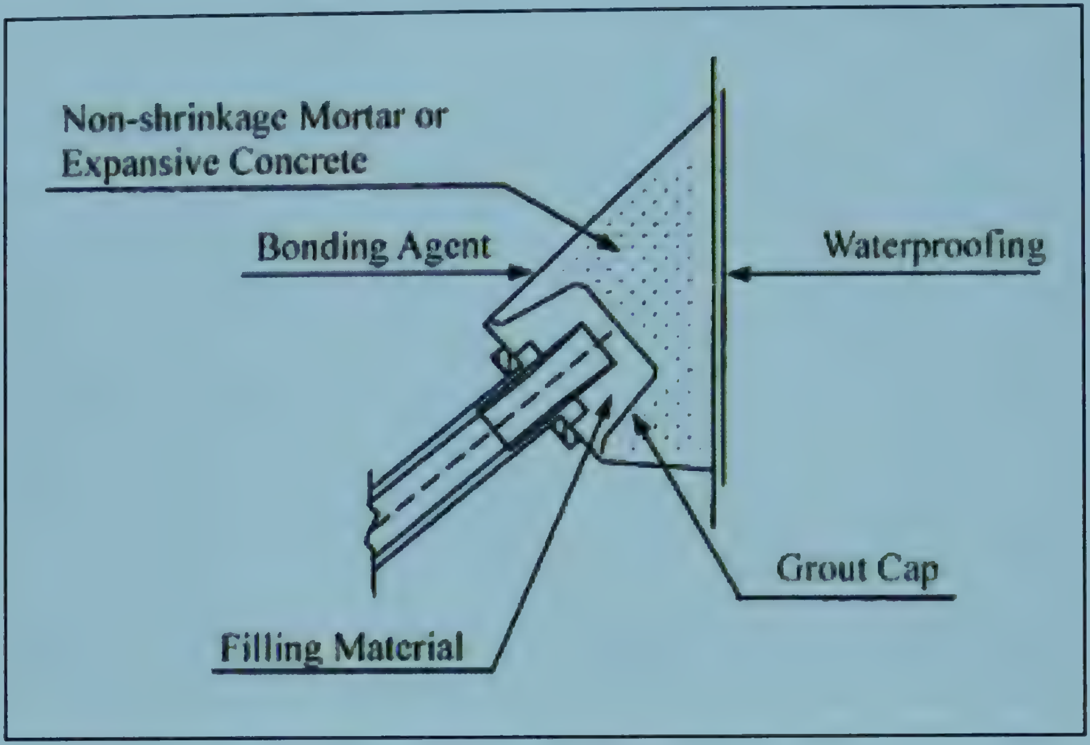


Fig.5.16 Notching Arrangement [2]

In case that the stay cables are anchored at the position which is made by cutting off a part of structure of pylon or girder, the notching configuration, **Fig. 5.16** should be determined with consideration of aesthetic. Moreover, the color adjustment of cover concrete should be considered if necessary. After prestressing, adjusting the prestressing force and grouting, the notching area shall be filled with non-shrinkage mortar or expansive concrete for being protected. Also, if necessary, providing additional reinforcement to prevent cracks on the surface, applying bonding agent and waterproofing should be done to assure rust prevention.

5.5 Construction of Stay Cable

This section describes the methods of construction of stay cables, their installation, stressing etc. for getting the general idea, and may vary from project to project depending on the contractor and supplier of the cables.

5.5.1 *Fabrication and Transportation of Stay Cables*

5.5.1.1 Generally, stay cable system consists of inclined cable (MTE), anchorage, stay pipe and filling material. Depending on whether stay cables are fabricated in a factory or at the construction site, the transportation method, placement and installation process vary.

Pre-fabricated stay cables usually fabricated as all one unit with HTS, stay pipe and filling material. Therefore, since it is difficult to adjust the length of stay cables at the construction site, it is important to determine the length of cable to be fabricated considering deflection of cable, and difference of as-build dimensions of pylon and girder from design documents. Site-fabricated stay cables can be assembled in parallel with construction of pylon and girder, and constituent material can be brought into construction site individually. The constructed area of girder or on the ground backside of abutment can be used as the space for processing the cable. However, because the space is limited and there is congestion of different work, it is necessary to plan and create an efficiency site layout and work schedule.

The use of site-fabricated stay cables which is different from conventional system is in vogue. For example, they are the non-grout type which does not require grouting because the cable is coated with zinc or resin, and the semi-fabricated type where the coated cable, stay pipe and filling materials are integrated in the factory and carried to the site. Since existing examples of these cables are few, it is necessary to fully understand the characteristics of the materials and system prior to construction.

5.5.1.2 Pre-fabricated cables are usually transported to the site, rolled up to the reels of 2.0 m to 3.0 m diameter. The size of the reel is, of course, depends on the diameter and length of the cable. Therefore, in case of the land transportation, it is important to consider the law restriction issue in advance.

5.5.1.3 It is important for the stay cables and anchorages to be protected from damages during loading, packing, unloading and especially during storage. For site-fabricated cables, it is necessary to pay attention to rust prevention because the retention time tends to be longer.

5.5.1.4 Stay pipe is used for the purpose to ensure the durability, rust prevention and overall protection. Its material can be HDPE pipe, FRP pipe and steel pipe. Therefore, suitable materials should be chosen depending on the cable fabrication and installation method and rust prevention method. In addition, in case of the use of long cable, it is necessary to consider that the connection and erection are safely done and winches have sufficient capacity to push and pull the cables.

5.5.2 *Installation of Stay Cable*

In recent days, majority of Stay Cable Bridge Projects in India are executed by using Parallel Strand cable systems. This system is executed by site fabrication by unwinding coils of strands and stressing. Below procedure may vary as per vendors.

5.5.2.1 Preparatory Works for Cable Assembly.

- **Storage of Components**

  All components for the cables shall be stored properly to prevent contamination and damages. All parts of the anchorages, especially the wedges, anchor blocks with ring nut and the passive anchorages shall be stored at a dry place and protected against contamination. All HDPE-pipes for the sheathing shall be stored properly, lying straight without being exposed to deformations in cross direction.

- **Working Platforms**

  Working platforms outside the pylon are required for assembly of the cables up to the end of the finishing works. At the deck structure, appropriate measures have to be taken to have access to the anchorage area. Platforms and scaffoldings need to be provided by the construction company with their required minimum dimensions in consultation with the vendor.

- **Equipment**

  General equipment like mobile cranes, forklifts, power supply, etc., has to be provided.

- **Anchorages Preassembly**

  Anchorages will be preassembled on site. Part of the preassembly is also the placing of the wax plate, shown in **Fig 5.17**.

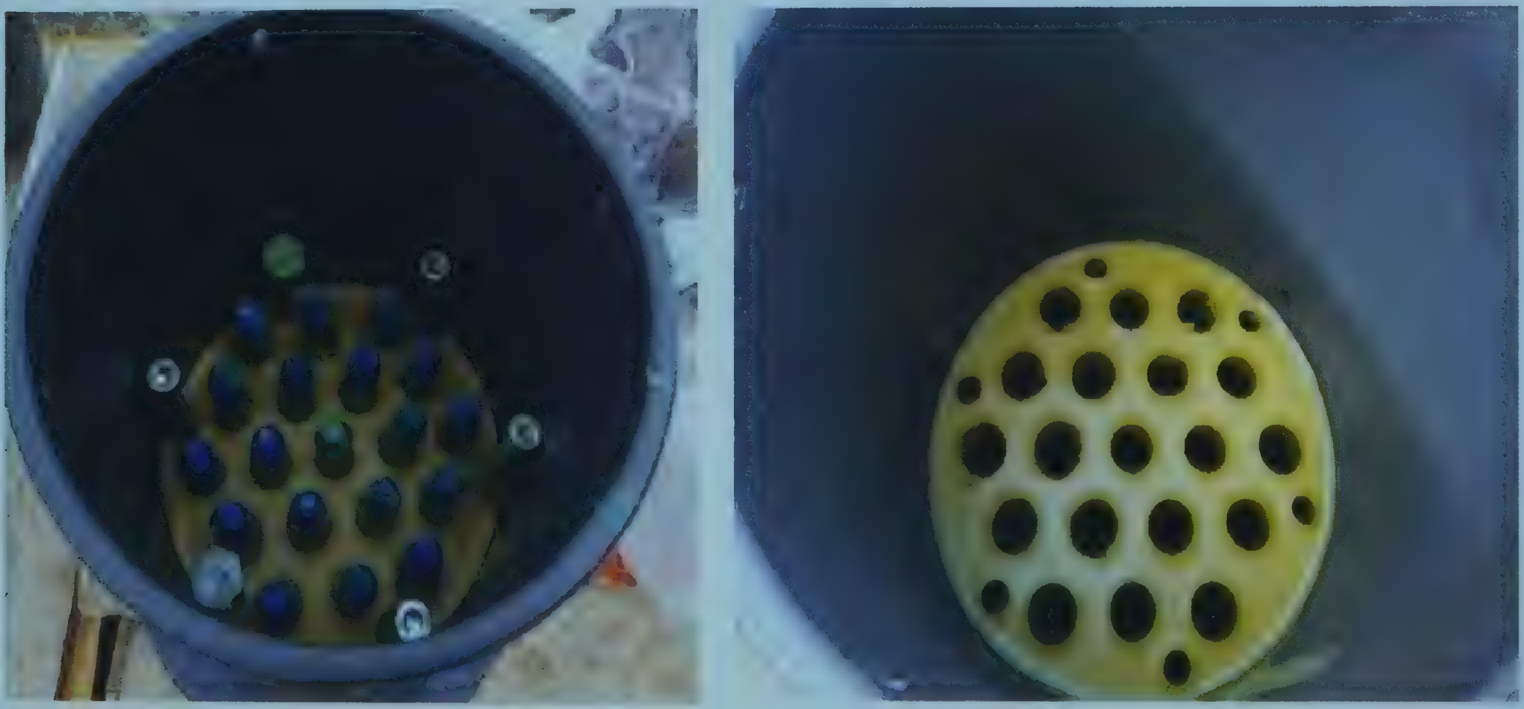


Fig. 5.17 Poured Heated wax into back of Anchorage once all holes are plugged to form the wax plate (Credits: L&T)

- **Installation of Active and Passive Anchorages**

  Prior to installation of the active anchorages, the trapezoidal thread of the anchor block that will not be accessible afterwards has to be protected, **Fig 5.18** with protection compounds:

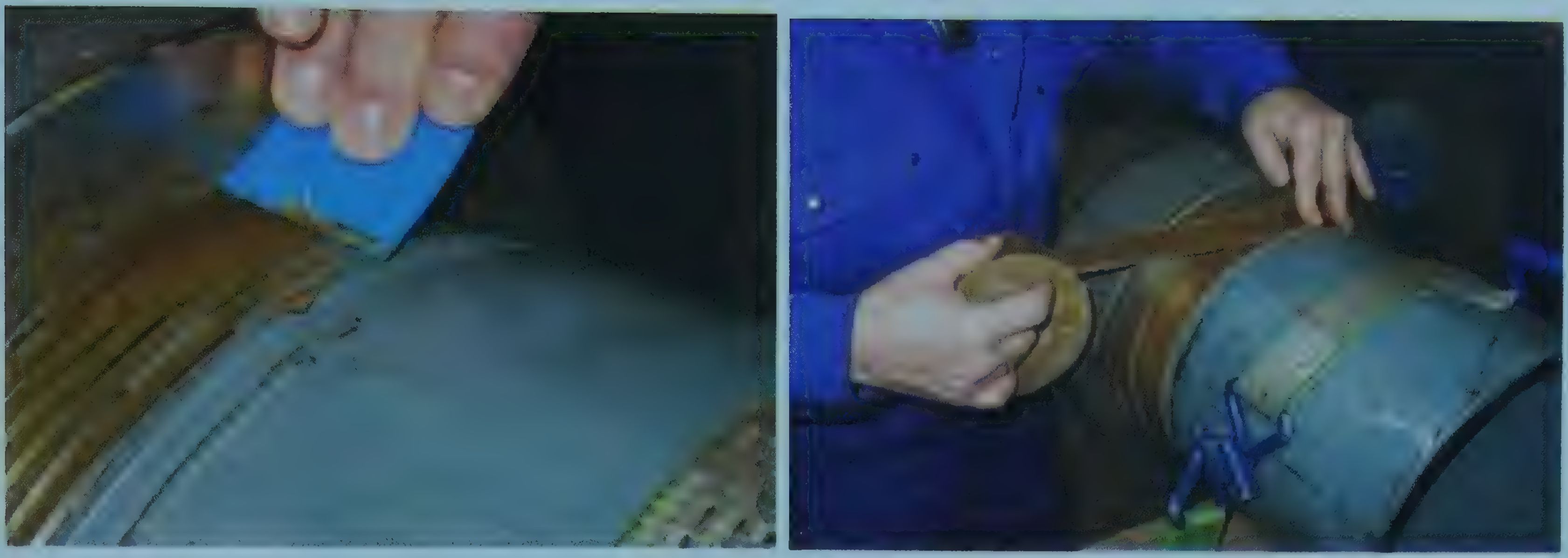


Fig. 5.18 Filling thread with FILL and Wrapping with Petrolatum Tape (Credits: L&T)

In most of the cases, active anchorages are installed at the superstructure, whereas dead anchorages are placed within the Link anchor boxes. Installation of dead anchorages shall be done prior to the installation of the anchor boxes within the pylon.

At deck side it is necessary to fix the anchorage temporarily to the tapped holes of the bearing plate, before it is held in place by stressed strands. If strands are not going to be installed shortly after placing the anchorage, the anchorage surface has to be protected by covers to protect it from corrosion and damage

The injection opening of the ring nut (on its circumference) must be covered by duct tape to protect it from dirt and corrosion. All other threaded blind holes on the ring nut or dead anchorage front surface, which are used for connecting the caps later on, shall be protected by duct tape as well.

- **Corrosion Protection of Wedges**

  Wedges have to be treated with corrosion protection compound prior to using them for anchoring the strands at pylon and deck side. The corrosion protection material is heated up within a melting pot and wedges that are placed into a sieve are immersed. After this, wedges are stored for drying.

- **Preparation of Cable Sheathing**

  The HDPE-sheathing supplied in straight single lengths is welded together to the required length with the help of a butt-welding machine. The length of the sheathing depends on the current temperature of the PE-tubes.

  A confined environment should be used for welding in order to protect the respective pipe sections from weathering and dirt during the welding process. This can be done using a welding container or a welding shed.

  After welding to its required length, the sheathing needs to be equipped with all necessary cable components and auxiliary tools.

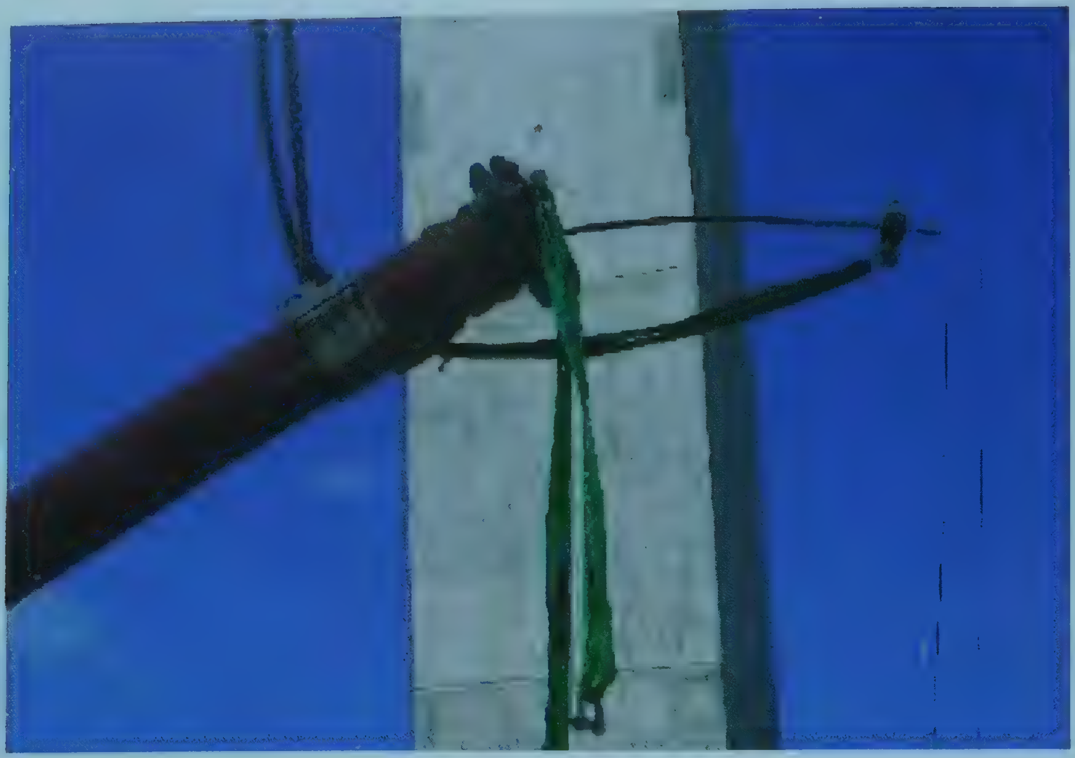


Fig. 5.19 Example of Strand Secured by Strand Clamp (Credits: L&T)

5.5.2.2 Cable Assembly

Once having prepared the sheathing with the strand secured by the strand clamp **(Fig. 5.19)** a crane lifts the upper end towards the anchor box at the pylon, **Fig. 5.20,** the sheathing is attached to the tube at the front of the Link anchor box using eye bolts. The first strand can be inserted and anchored within the dead anchorage. At deck the lower end is held up by a fork lift or similar to prevent the sheathing from being scratched on the ground. By using the forklift, the sheathing is positioned in front of the deck recess pipe and the first strand's end is pushed through the recess pipe and anchorage hole using a PE hose for guidance.

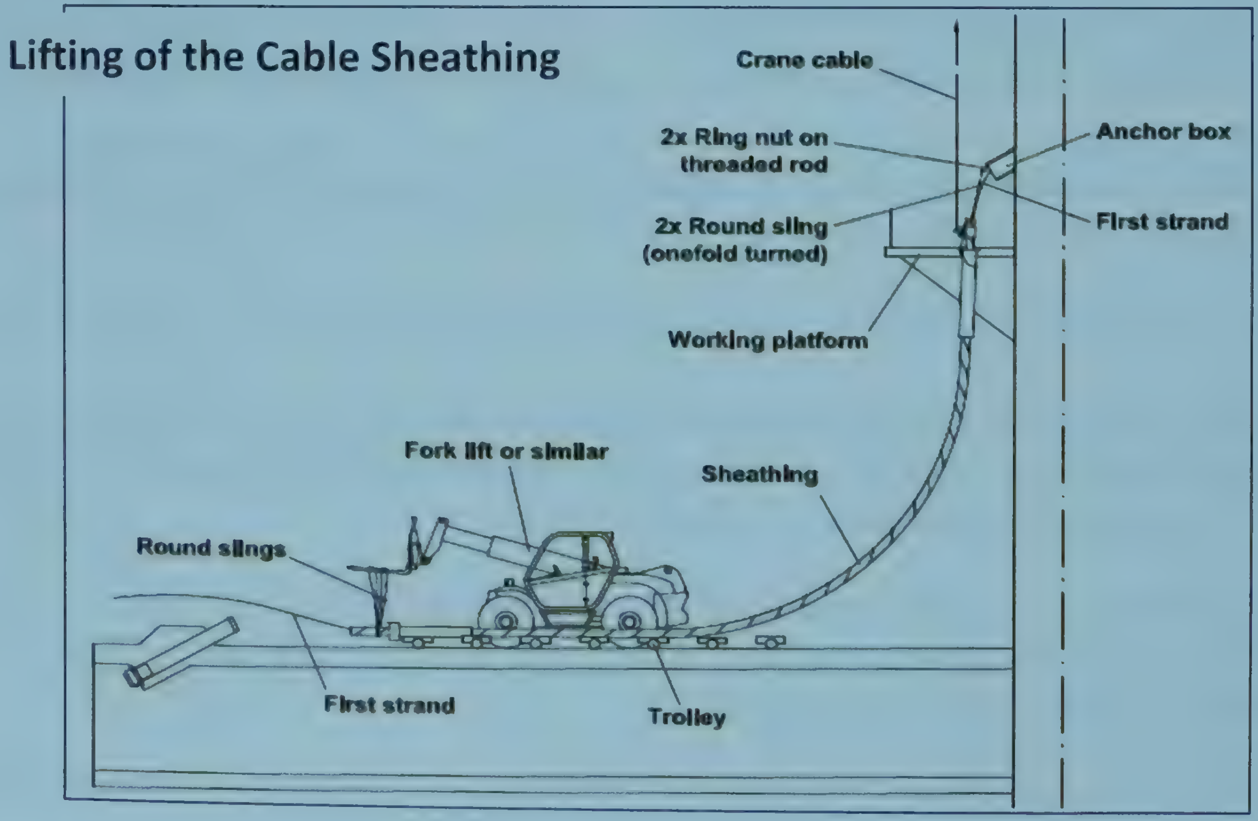

Lifting of the Cable Sheathing
Crane cable
2x Ring nut on threaded rod
Anchor box
2x Round sling (onefold turned)
First strand
Working platform
Fork lift or similar
Sheathing
Round slings
First strand
Trolley

Fig. 5.20 Lifting of the Cable Sheathing

The strands are supplied reel-less and have to be placed in proper un-coilers. They will be cut to length prior to installation. Using a hydraulic pusher every strand will be pushed into the sheathing and be installed and stressed according to the **Fig. 5.21**.

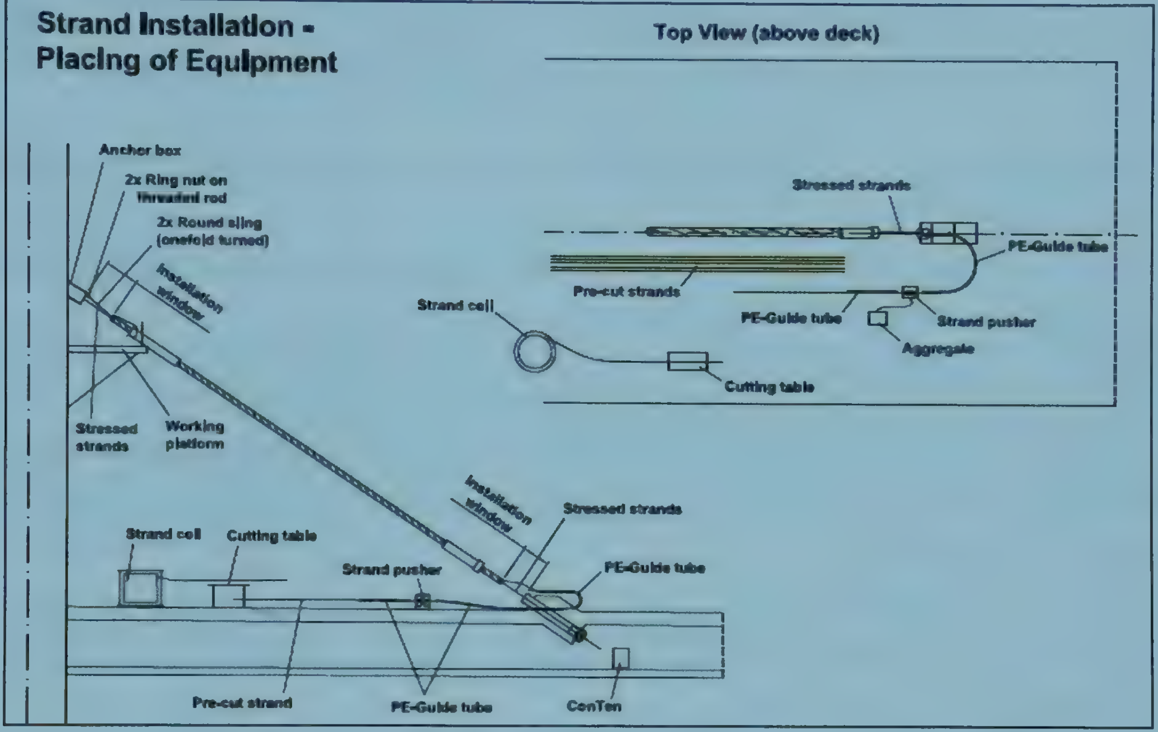

Strand Installation - Placing of Equipment
Top View (above deck)
Anchor box
2x Ring nut on threaded rod
2x Round sling (onefold turned)
Installation window
Stressed strands
Working platform
Strand cell
Cutting table
Strand pusher
Installation window
Stressed strands
PE-Guide tube
Pre-cut strand
PE-Guide tube
ConTen
Stressed strands
PE-Guide tube
Pre-cut strands
Strand cell
PE-Guide tube
Strand pusher
Aggregate
Cutting table

Fig. 5.21 Strand Installation – Placing of Equipment

Cable Lifting and Stressing of 1st Strand

The following operations are necessary:

i) The crane cable is attached to the pipe clamp and the sheathing is lifted towards the anchor box anchorage **Fig 5.22**. There the round slings with shackle are attached to the tip of the Link anchor box by eye bolts, a chain hoist can be used to adjust the position of the sheathing.

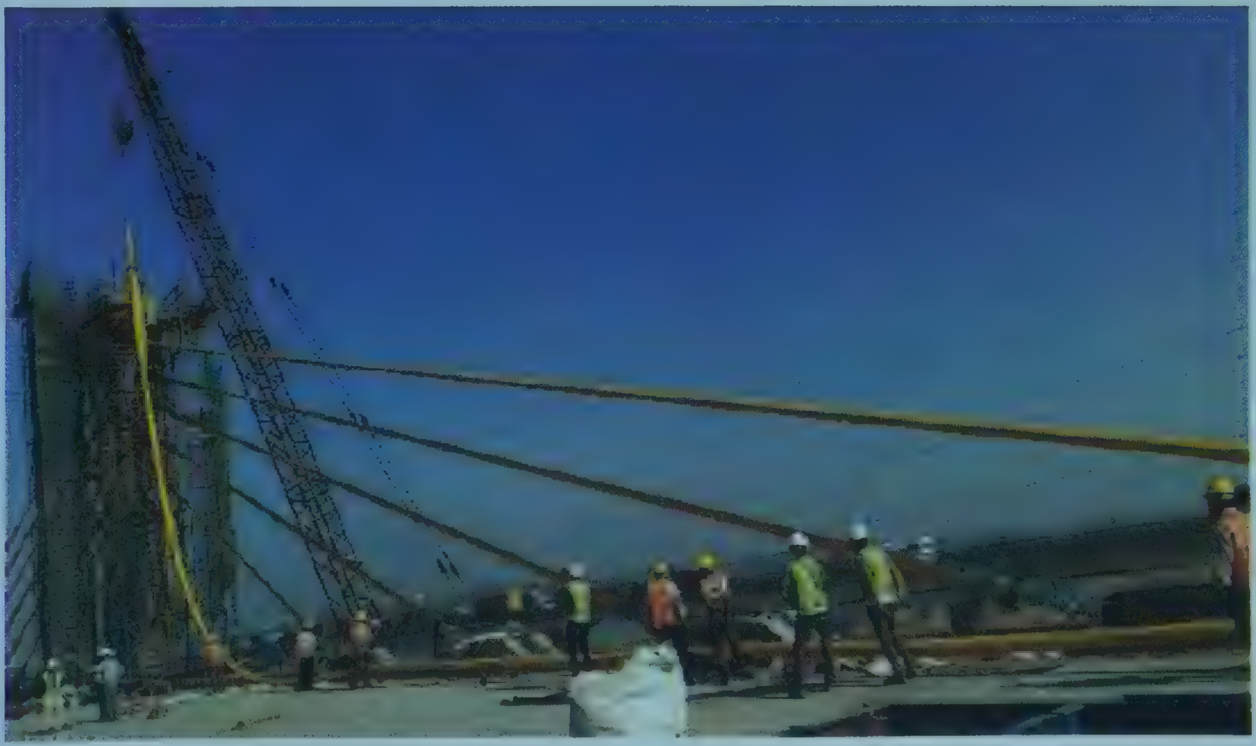


Fig. 5.22 Lifting of HDPE Pipe (Credits: L&T)

ii) By gently lowering the crane cable, the round slings take over the weight of the sheathing.

iii) At deck side, a fork lift or similar, supports the deck end of the sheathing and prevents it from being scratched on the ground and from kinks in the first strand caused by the weight of the sheathing.

iv) The deck end of the first strand is fed through the PE guide tube into the hydraulic pusher.

v) At top, the strand clamp is removed from the first strand, which is positioned in front of the anchor box.

vi) The pusher is used to move the strand upwards into the correct hole of the anchorage until it protrudes the anchorage by 70 mm.

vii) At this point a wedge is placed on the strand and fastened by applying three hammer strokes to a short pipe placed around the strand.

viii) At deck side the strand is removed from the pusher and inserted into the anchorage with the help of a PE-feeding hose that is pushed upwards through the corresponding cone hole.

ix) A wedge that is protected with corrosion protection compound has to be pushed onto the protruding strand together with a steel compression tube and must be placed in front of the cone hole.

x) The strand can then be stressed according to the stressing protocols

xi) After the strand has been stressed, the crane cable can be removed and installation and stressing of further strands may continue.

5.5.2.3 Stressing

The strands of the Stay Cable are stressed one by one. This will be done using any of the pre stressing system provider's patented system. Cable force may also be adjusted with the help of the gradient jack after installation of all strands.

The stressing records are the basis for all stressing operations. They are prepared in advance with all necessary data given by the structural engineer considering the deformation of the structure as well as the forces, which have to be applied. In order to obtain a most accurate application of the stressing forces, it is recommended to carry out at least two stressing steps. In doing so, force differences in single strands due to differences in temperature during installation can be equalized.

- **Stressing of Strands**

  For stressing of Multistrand Stay Cables, two different methods are available;

  1. Single stressing system with mono-jacks
  2. Multistrand jack stressing (gradient jack)

  Appropriate stressing records for every single cable will be generated and submitted for approval with the data provided by the structural engineer in advance. Needed data are cable forces prior to and after stressing as well as the coordinates of center of the bearing plates (measured on the outer surface) before and after stressing. The procedure and principles of iso-tensioning is explained in separate section of stay cable systems.

- **Verification of Forces**

  The most important and common method for cable force verification is doing Lift-Off-tests with the mono-jack. Thereby a minimum of three strands of one cable will be lifted until the wedge is moving. With this actual pressure inside the jack, the force can be calculated.

5.5.2.4 Finishing works

***Installation of Dummy Strands**

Because the anchorages are not always fully occupied with the maximum number of strands, the free conical bore holes are filled with a short dummy strand and a wedge. Also within the steel bearing pipes at deck and pylon dummies have to be placed. Strand occupancy drawing is given by engineer in general, with detailed information on location, lengths and preparation of dummies.

- **Corrosion Protection Works at the Anchorages**

  At active anchors, the ring nut threads are injected as well as the short caps at dead end side. At stressing anchorages, the strand protrusions are covered by PE-protection tubes (filled with corrosion protection compound) is installed without being filled.

- **Activating the Sealing**

  The Threaded rods must be tightened applying a certain torque as per vendor's guidelines.

  The interspaces between conical boreholes, wedges, and strands are automatically filled by corrosion protection wax because the wax plate that has been inserted into the anchorages before will be compressed while activating the sealing unit.

- **Protection of Ring Nut Thread and Anchor Block Thread**

  Corrosion protection compound must be injected into the ring nut thread with a grease pump via a grease nipple at the outside of the ring nut until the compound exits at the thread run-out. After injection, a hexagonal socket plug with conical thread permanently replaces the grease nipple. The corrosion protection of the anchor block thread in front of the ring nut is the same as behind, which was applied during anchor preassembly, and consists of a FILL material to be filled into the thread, and an FEU-tape to be wrapped around the thread afterwards.

- **Assembly of Caps at Stressing Side**

  The PE-Protection tubes have to be cut to the required length according to the strand protrusions. Corrosion protection compound must be filled, which are then pushed over the strands with the filled end facing towards the anchorage, until they rest against the anchorage. The tubes have to be sealed with plugs at the rear side. Place the Cap with the Sealing ring on the ring nut and tighten the hex nut screws.

- **Assembly of Caps at Dead Anchorage Side**

  The Foam lining has to be fixed to the inside of the Cap with adhesive glue. Remove the foam lining in front of the vent opening at the cap's back. Place the Cap on the anchor surface and tighten the hex nut screws, constantly and crosswise. The torque to be applied is as per system provider's specifications. From the bottom the cap is filled with corrosion protection compound until it exits at the vent opening on top. Take off all injection equipment and place plugs at both tapped holes

- **Installation of Steel Bearing Pipes**

  After completion of installation and stressing, the strand bundles have to be compacted by a special compaction clamp to be able to install the steel bearing pipes. At deck, the steel bearing pipes have to have a distance as per vendor's specifications away from the recess pipe, while the respective distance at the anchor box measured from the end of the connection tube at the anchor box can be seen.

  At deck side, installation of steel bearing pipes must not be done until all stressing works with jacks are finished completely. Compact strands by means of the strand compactor, which therefore is located between recess pipe and designated location of the steel bearing pipe. Position the two parted steel bearing pipe with PE linings inside fix the 4 screw fittings with following torques specified by the vendor. The strand compactor is removed.

5.5.2.5 *Closing of the Sheathing*

- **Temporarily**

  Right after strand installation and also anytime the installation process is interrupted for not only a short period of time, the installation windows shall be closed / covered either by moving the sheathing and sleeve or by using tarpaulin. With this, ingression of large amounts of water can be avoided during construction phase.

- **Permanently**

  After installation of all cable components, the sheathing is lowered onto the recess pipe. Slowly release the sheathing from its position by using a chain hoist in between round slings and the upper Link anchorage at the pylon. Lower the Connection part onto the O-Ring AV and onto the deck recess pipe flange. The sheathing is protected from being pushed upwards by 4 bolts M16 in the connection part. Fix the PE-sleeve's flange between the end of the bearing pipe of the anchor box and Flange Tube Anchor Box with Rubber Ring and the Flange Ring Sleeve, by using four bolts of M16.

5.6 Quality Control during Construction

5.6.1 *Accuracy Management of Entire Bridge*

In the cantilever construction of cable-stayed bridges, the management is necessary not only to ensure the determined configuration of bridge such as vertical alignment in completion stage with a specific accuracy range, but also to control the deflection of girder, prestressing force of stay cable and stress of member in each construction stage. In addition, special attention is important for accuracy management of inclination of pylon. Since deflection of girder, inclination of pylon, prestressing force of stay cable stress of member all have a close and interdependent relationship mutually, a comprehensive management method is essential. In contrast to other girder bridges, cable-stayed bridges have less stiffness, and the prestressing force of stay cable and deflection of girder are more influenced by change of temperature. Therefore, it is important to carry out the accuracy management with clearly grasping the margin for the prescribed limit values in each stage of construction.

In addition, as concerns the cable-stayed bridges, the expansion of girder is also an important issue in accuracy management. The amount of expansion includes the elastic contraction due to prestressing force of stay cable, the contraction caused by creep and shrinkage, the expansion and contraction by change of temperature. In particular, it is necessary to consider in advance the manufactured length of cables. Furthermore, if the width of girder is wider, deflection control in the transverse direction may be required. Also, in case of skew bridge and curved bridge, torsion control may necessary. Thus, these cases also need consideration.

The construction management of cable-stayed bridges may need tension adjustments after the closure of center of span. Although there is often a difference between the calculated deflection and the actual deflection, as concerns the cable-stayed bridges, it is adjustable by adjustment of cable force not only during cantilever erection but also after closure.

The following items are usually included in the construction management of cable-stayed bridges~~.~~

- ✓ Preparation of a detailed data base; detailed execution plan (deflection, expansion and contraction) on each construction stage and calculated value influenced by various factors.
- ✓ Measurements at the construction site ; clearly grasping the deflection of girder, inclination of pylon, prestressing force of stay cable and stress of member at every stage of construction.
- ✓ Data processing and analysis; analyze the cause of difference comparing measurement values collected at the construction site with design planned values. The influence values are reviewed and any relevant adjustments of prestressing force might be made.

As the above items are done at the construction site, and the amounts of data to be dealt with are large and complex, it is standard to establish a management system where computations on the basis of data is made available to site for further action at the earliest.

The basic items measured at the construction site are shown in the **Table 5.1**.

In the actual construction of bridge, the followings are thought as factors which cause the difference comparing with design planned values: variation of material characteristic, difference in estimation of shrinkage and creep, difference in loading of construction equipment, difference in deflection and elongation by temperature, and other factors in construction. In order to minimize the difference possibly caused by these factors and to ensure the configuration and stress condition as designed in completion stage, it is important to use the construction management system and enhance the management quality for each construction step.

Table 5.1 Items to be measured in Construction Management System [2]

Item	Place	Measurement Objectives
Deflection	*Girder*	Obtain data on deflection of the girder during construction to be used for deflection management.
Expansion	*Girder*	Regards the expansion amount of girder to use in length control.
Tension	*Girder*	Used for tension management during cable tensioning. Get data on tension changes during construction to be used for tension management.
Temperature	*Girder* *Pylon* *Cables Air Temperature*	Grasp member temperature changes caused by sunshine and investigate the effect of temperature on girder deflection and cable tension, gather data for correction.
Stress	*Girder* *Pylon*	Obtain data on stress changes on the main girder and tower during construction to be used for management.
Inclination	*Foundation* *Pier* *Pylon*	Obtain data on foundation, bridge pier and pylon inclination caused by unbalanced moment, investigate their effects on girder deflection.
Settlement	*Foundation*	Obtain data on foundation, caused by incremental loading, unbalanced moment, investigate their effects on girder deflection.

It is necessary to determine the range of management values for construction management in terms of not only structure safety in construction or completion stages, but also serviceability during in service stage and stress condition of members. **Table 5.2** shows examples of general management indicators for the main girder, pylon and stay cables. In addition to checking the indicators and confirming them with the design conditions and construction standards, it is necessary to consult with the owner of the bridge to confirm the process.

Table 5.2 Construction Management Indicators [2]

Deflection of Girder	The level which does not produce any problem for vehicle running during the service time.
Inclination of Pylon	The level which does not produce any trouble due to additional moment.
Introduced Cable Tensions	The level in which member stress is within the limit values

5.6.2 *Quality Control of Concrete*

Generally, very high-strength concrete is used for cable-stayed bridges and extradosed bridges. Therefore, thermal cracking should be investigated in relatively massive parts such as anchorage of stay cable and base of tower.

Moreover, it can be considered that crack occurs in girder during cantilever construction because the existing slab deck restrains the new slab deck's thermal and shrinkage deformations. Since the girder of cable-stayed bridges and extradosed bridges usually has wide cross-section, this phenomenon becomes more pronounced. Therefore, reinforcement should be arranged based on thermal stress analysis carried out beforehand.

In the girder, the HTSs and reinforcing bars at the anchorage of stay cable and top of column are placed in highly condensed arrangement, and the concrete is usually placed by using concrete pumps. Thus, adequate slump should be determined by considering the filling and workability of concrete.

Since the reinforcement of the pylon, where the stay pipe for stay cable anchorage or saddle, reinforcing steels, steel frames for assembly are arranged, are highly condensed and concrete generally need to be pumped to high altitude, the slump that is necessary for pumping performance should be determined. Also, concreting method shall be examined by considering sufficiently the capability of carrying facility based on the relationship of factors such as characteristic of concrete, height of pylon, quantity of one-time placed concrete. If necessary it is advisable to do initial mockups to study the construability of the elements.

5.6.3 *Quality Control of Steel Members*

With regards to quality control of steel member, for example, in case of composite bridge with steel girder beams with concrete deck, or mixed girder where steel girder is used in the center span, or pylon's anchorage structure made of steel shell, or a steel deck or steel pylon itself, reference should be made to *IRC:SP:104. "Guidelines for Fabrication and Erection of Steel Bridges"* Moreover, in usage of steel members, consideration is necessary to ensure accuracy required by trial-assembly works in the factory and pre-assembly at construction site prior to erection.

5.6.4 *Prestressing Management of Stay Cable*

Stay cable alignment has very small change of angular which is similar to external cables, and loss of prestressing force by friction is also small. Therefore, as a general rule, the method to control relation between cable force and elongation should be applied for the prestressing management of stay cable. In addition, prestressing management requires the sufficient understanding of the characteristics of anchoring system of stay cable (anchorage type and saddle bent type) and method of fabrication (site-fabricated and pre-fabricated).

5.6.4.1 Prestressing Management for Separate Fixing type.

In the prestressing management for separate fixing type, prestressing management should be done dealing with tension force primarily while elongation secondary. There are two methods for initial tensioning. In one method, all PC strands are inserted at once and tensioned all together using center hole prestressing jack. In other method, PC strands are inserted and tensioned one by one using hydraulic single strand jack. After the initial prestressing during construction, it is necessary to adjust the tension appropriately in order to improve stress conditions of girder, pylon and pier, and to correct the final configuration of the structure. Generally, hydraulic jacks are placed in symmetrical position in the pylon or girder. Moreover, cable tension can be measured by using load cells that are attached in each cable. And also, it is found indirectly from the vibration frequency of cable obtained by using accelerometer to vibrate cable intentionally.

5.6.4.2 *Prestressing Management for Through Fixing Type.*

For prestressing management for through fixing type, since the tensioning is conducted from both ends of the girder and loss of prestressing force by friction (μ is small, prestressing management should be primarily done with tension force while elongation is secondary. However, the form and friction coefficient of the saddle shall be considered in calculation of elongation. The Influence of this Friction Coefficient can follow *"Guidelines for the use of External and Unbonded Prestressing Tendons in Bridge Structures (IRC:SP:67)".* Generally, prestressing should be performed collectively so that elongations at both ends are equal in order to apply the required cable force to all cables that are used in the same stage of installation. In this type of cable, if designed tensions at both sides of saddle are not same, the slipping of cable at the saddle should be checked and the required tension should be secured.

5.6.5 *Acceptance Inspection and Storage Management Plan of Materials*

Materials that are used for stay cables shall be checked whether it satisfy the required standards and specifications on acceptance. Also, scratches, twists, rusts shall also be checked by visual inspection. Therefore, it is necessary to establish management plan regarding acceptance inspection of material and storage procedure prior to the commencement of construction.

Since the saddle and steel anchorage structure require high accuracy in fabrication, quality control plan shall be prepared prior to fabrication, and acceptance inspection of material including factory inspection shall be performed.

In addition, the "Recommendations for Stay Cable Design, Testing and of U.S. Post-Tensioning Institute (PTI) can be referred. This document presents considerations regarding on-site acceptance inspection and quality control of components of stay cable.

- ✓ The components of site-fabricated cable; diagonal cable (HTS), anchorage, stay pipe and filling materials, are transported separately to the site in accordance with the progress of installation. Therefore, acceptance inspection for each lot is executed immediately after arrival of materials. The transported materials are inspected by collating the mill certification report, delivery statement and the detailed order sheet. Also, acceptance checklist is used to confirm that scratches, twists, rust are not found. The results are saved after recording it in a check sheet.
- ✓ Adequate storage and curing should be done in order not to impair the quality of materials during storage. Since these materials are generally heavy and stay cables are packed in drum form, it is necessary to maintain safe storage conditions by considering over-turning and rolling during temporary setting.

CHAPTER - 6

LOADS AND LOAD COMBINATIONS

SECTION I: LOADS

The description of actions, their classification, nomenclature of design situations and combination for verifying different limit states as explained in IRC:112, are included in this guideline. The loads and actions on stay cables and on account of the stay cables are based on the Fib 89 **[1].**

6.1 Dead Load

Dead loads shall be in conformity with the provision of IRC:6. The unit weights of the material shall be in accordance with the provisions of Clause 203 of IRC:6.

Stay Cable Weight

For stay, the weight of the main tension element, and any additions to the stay, such as sheaths, grout, etc., shall be included as dead load in the stay. The Self weight of the stay cable and its connection including sheathing shall be as per the recommendations of the manufacturer.

In the absence of the any data following can be used:

Weight of parallel wire / parallel strand = $A_s \cdot n \cdot \rho_s$

where

A_s - Nominal area of wire / strand,

ρ_s - Density of the steel 7850 kg/cum

n – number of wires / Strands

The Weight of the protection material viz. Heavy galvanization, Sheath, grout can affect weight of the cable and increase its weight by 10% to 25%. **Fig. 6.1** shows example of a 7-strand prefabricated cable (GJ15-7), having weight 9.79 kg/m (i.e. about 25% additional weight of protection)

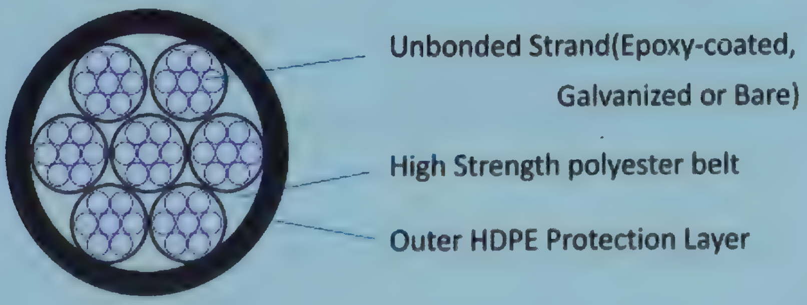

Unbonded Strand(Epoxy-coated, Galvanized or Bare)
High Strength polyester belt
Outer HDPE Protection Layer

Fig.6.1 Strand Prefabricated Cable (GJ15 - 7),

6.2 Prestressing Force in Concrete

For analysing the structure, prestress, which has time dependant variation, is considered as a force acting on concrete elements. Two characteristic values of prestressing force shall be used, γ_{sup} (i.e. higher estimate) and γ_{inf} (i.e. lower estimate).

6.3 Secondary Effects

In the design of cable-stayed bridges the effects of shrinkage and creep of concrete shall be considered as per the IRC:112 and load factors and combinations as per IRC:6.

6.4 Superimposed Dead Load

Loads from handrail, crash barrier, bridge furniture, footpath and actual or provisional loads from utility services, etc. are covered under this loading. Partial safety factors and load combinations shall conform to provisions of IRC:6.

6.5 Surfacing and Wearing Coat

Different partial load factors are to be used depending upon whether the load is adding to the effect of variable loads or relieving. This is to account for re-surfacing, with or without removing existing coat, and possibility of change in type of surfacing. Partial safety factors and load combinations shall conform to provisions of IRC:6.

6.6 Snow Load

The effects of Snow Load, wherever applicable, shall be considered as per provisions of IRC:6.

6.7 Vehicular live load (Q)

6.7.1 *Vertical Loads Effects*

Following shall be considered for Vehicular Live Load:

i) Vehicular Live Load in accordance with **IRC:6 Clause No. 204.1**, including combinations of Live load according to Clause No. 204.3 shall be considered for the design.

ii) Vehicular Live Load with Congestion factor will be considered according to **Clause No. 204.4** of IRC:6. For congestion factor loading to be applicable, all the spans shall be loaded.

iii) Special Vehicle Loading shall be considered in accordance with **Clause No. 204.5** of IRC:6.

iv) Reduction factor for the Vertical effects of the live load shall be considered as per **Clause No. 205** of IRC:6.

6.7.2 *Impact factor due to Vehicular Live Loads*

Vehicular Live loads are multiplied with an Impact Factor to accommodate this dynamic effect in the design of bridges. The magnitude of the impact factor depends on the bridge span, stiffness and surface roughness, and vehicle dynamic characteristics such as moving speed and isolation system.

Impact factor shall be Considered According to **Clause No. 208** of **IRC:6**.

6.7.3 *Longitudinal Forces*

Longitudinal force caused by tractive effort of vehicles or by braking, and / or Longitudinal force caused due to frictional resistance offered by free bearings due to change in temperature, or due to creep or shrinkage, shall be considered in accordance with **Clause No. 211** of IRC:6.

6.7.4 *Centrifugal force of Vehicles Travelling at High Speed along a Curved Path*

Shall be considered in accordance with **Clause No. 212** of **IRC:6**.

6.8 Pedestrian Load/Foot Path Load

Shall be considered in accordance with **Clause No. 206** of **IRC:6**.

6.9 Fatigue Load

The performance of Stay cable is dependent on its fatigue performance under repeated live load cycles. However, this will depend on the magnitude of the live load as a proportion of the dead load.

For all concrete / steel components of the cable stayed bridge, excluding the stay cables and its connections, provision of IRC:6 for fatigue load and relevant annexure of IRC:112, provision of IRC:22 and IRC:24 shall be applicable. For stay cable design, provisions given herein shall be followed.

All cable stayed bridges need to be designed for fatigue limit state as explained in **Chapter-7** for design.

Fatigue Truck to be considered shall be in accordance with **Clause No. 204.6** of IRC:6. The stress range resulting from the single passage of the Fatigue load along the longitudinal direction of the bridge shall be used for the fatigue assessment with the fatigue load so positioned as to have worst effect on the detail or element of the bridge under consideration. In case of divided carriageway, single passage of fatigue load shall be considered for each carriageway. The Fatigue truck shall be placed on the carriageway with respect to the kerb in accordance with IRC:6.

6.10 Thermal Loads

All steel and concrete bridges shall be designed for IRC:6 requirements for uniform temperature ranges and thermal effects. In addition, non-uniform temperature variations shall be considered. Preference shall be given to using a light colored HDPE pipe.

A temperature difference between the HDPE or other sheathing and the MTE (Main tension Element) may affect the design of any expansion joint details for the PE pipe.

Following Thermal effects shall be considered for design:

i) Uniform Temperature rise/fall as per IRC:6.

ii) Thermal gradient through the deck (Positive and Reversal Gradient as per IRC:6).

iii) Differential temperature between various members of the structure, especially that between the cables and the rest of the bridge, must be considered in the design.

For temperature difference between cables and deck or between cables and pylon, values of 10°C and 20°C shall be used for brightly coloured HDPE pipe, and black HDPE pipe, respectively. Similarly, 5°C linear temperature difference between the opposite faces of the pylon shall be also considered.

All the above effects should be considered simultaneously or independently whichever is unfavourable for SLS checks. For ULS checks gradient effects across the section shall be neglected

6.11 Cable replacement

The design of cable-stayed structures shall provide for the replacement of any individual cable with a reduction of the live load in the area of the cable under replacement. The closure of the traffic lanes nearest to the cable shall be considered for the design. Some cable systems offer possibility of replacing cables strand by strand. In such situations, replacement method shall be clearly indicated in the Maintenance Manual.

6.12 Cable Breakage

All cable-stayed bridges shall be capable of withstanding the accidental loss of any one cable without the occurrence of structural instability.

According to SETRA **[2],** the impact dynamic force resulting from the sudden rupture of a cable shall be 1.5 (Dynamic Amplification Factor) times the static force in the cable. DAF of 1.5 is recommended for parallel strand cable and parallel wire cables. This force shall be applied at both the top and bottom anchorage locations in direction opposite to the cable tension.

Cable breakage shall be considered as Accidental load for design verification. Forces in other cables under this situation shall be limited to 0.8 F_{GUTS}.

6.13 Transverse Loads applied from the cables to the structure

Based on Fib 89 **[1]**, Guiding devices installed near the cable anchorages or saddle exits, laterally support and limit the transverse displacements of cables at this location. As a consequence, these devices protect the cable anchorages from the effects of transverse loads and the tensile elements from excessive angular deviation α_{SLS}.

The transverse support provided by the guiding device to the cable causes a kink in the cable geometry, **Fig. 6.2**, and a corresponding transverse load is applied by the cable on the guiding device as well as the structure supporting the guiding device.

The structure supporting the guiding device needs to be designed for the maximum transverse forces, F, applied by the cable. These transverse forces are the product of angular kink and axial cable force T_{SLS}, usually the maximum force during construction and service limit states.

For detailed design, angular deviations shall be considered from the following:

- Deformations of the structure and change of cable sag due to construction loads, wind and traffic loads, and due to temperature changes in the service condition
- Cable vibrations
- Construction tolerances

For preliminary design of the structure supporting the guiding device installed in a cable, an angular kink of $\alpha = \pm 1.4$ degrees (± 25 mrad) is suggested as a reasonable assumption for SLS. Careful detailing for connection of the guide pipe and its extension is required to allow the connection of both pipes considering the construction tolerances for the alignment of the guide pipe and the actual catenary profile.

For systems without a guiding device, care should be taken that the cable will not touch the guide pipe at excessive angular deviations, including construction tolerances, and that the anchorages are designed to resist the maximum transverse load as mentioned above. The lateral load shall be applied to the cable anchorage at the relevant location.

Also other transverse loads, such as the weight of cable pipe including vandalism protection and loads from internal dampers need to be considered.

The transverse force can be computed as follows:

6.13.1 *Ultimate Limit State*

$$F_{ULS} = \gamma\, \alpha_{SLS} F_{SLS}$$

where,

α_{SLS} corresponds to ± 25 mrad

γ - Load Factor 1.5

F_{SLS} – Max Tension in cable under Rare Combination

6.13.2 *Serviceability Limit State*

No service criteria is specified unless the guide pipe constitutes a support for a damper, in which case, maximum deflection of the guide pipe under SLS shall be defined by the damper supplier.

6.13.3 *Fatigue Limit State*

The following can be considered:

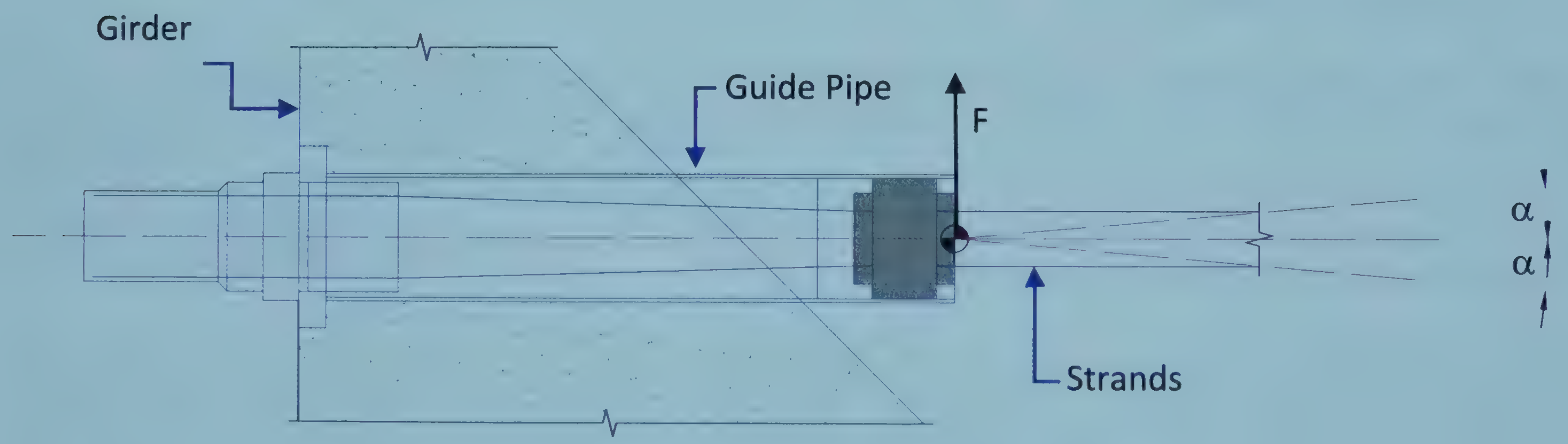
$$F_{FLS} = \Delta\alpha_{FLS} F_{perm}$$

where,

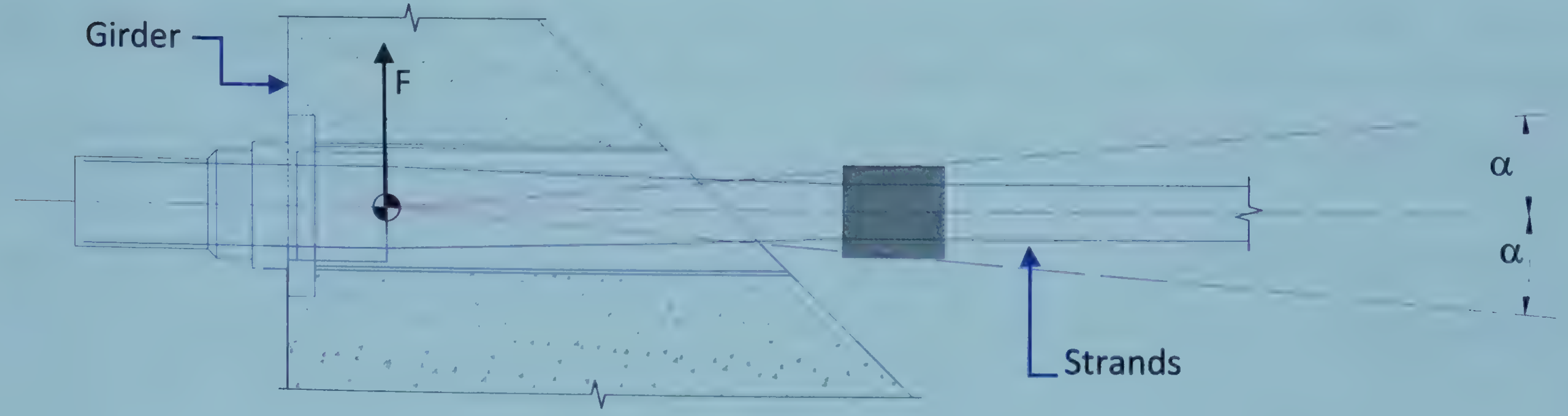
F_{perm} Tension in cable corresponding to the service loads under frequent load combination

$\Delta\alpha_{FLS}$ corresponds to ± 10 mrad

When the guide pipe constitutes only a support for a damper this value may be reduced according to the type of damper.



a) Elevation of Anchorage System with Guiding Device



b) Elevation of Anchorage System without Guiding Device

Fig. 6.2 Transverse Force by Cable Angular Deviation

6.14 Bending Stresses in Cables

Fib 89 **[1]** provides guideline for considering bending stresses in cable. Depending on the actual cable force and cable weight, and in the absence of other loads, the cable will take a specific catenary cable profile. The cable will be subjected to pure axial tension only if the anchorages at either cable end are placed along the tangent to that specific catenary cable profile. Any deviation of the actual cable profile from the above, due to:

- transverse displacement of the cable under loads applied to the cable or structure
- rotation of the anchorages relative to the tangent because of applied loads, temperature changes or installation tolerances of anchorages, bearing plates and guide pipes
- cable vibrations
- change of sag of cable due to change of cable force

will introduce local bending stresses into the cable, in the region where the transverse displacement or rotation is applied. Devices such as cable anchorages installed with an inevitable placement tolerance, guiding devices, saddles, and clamping devices along the cable length, e.g. for cross ties, introduce such deformations and corresponding bending stresses.

The cable rotations at the anchorage and saddle exit shall be calculated by the designer. When using cable systems designed and tested in accordance with Fib 89 **[1]** and this guidelines, fatigue angular deviations applied at the anchorage or saddle up to ± 0.6° (± 10 mrad) with 2 million cycles and service angular deviations up to ± 1.4° (± 25 mrad) with 100'000 cycles are covered by the system design and testing, and do not need to be considered in the design. Consequently, it is not required to add bending stresses to axial stresses.

For structures with larger angular rotations, the designer has to specify the usage of a guiding device or specify suitability testing, including fatigue, service and strength, with the larger rotations.

6.15 Fire, impact, vandalism

Design for fire protection if required shall be based on qualification testing of fire-resistant stay cable systems which protect cable from reaching a temperature greater than 300^{O} C for a period of not less than 30 minutes. The Fire Condition shall be considered as Accidental Event with cable dynamic force = 0.0 (Cable Loss from fire is assumed to be gradual).

6.16 Wind Loads

For wind load and design provisions refer **Chapter 9** and **Chapter 2** of this guideline.

6.17 Seismic Loads

For seismic design provisions refer **Chapter 8** and **Chapter 2** of this guideline.

6.18 Cable forces

The cable forces are of two types i.e,

1) cable force at the end of the staged construction which include dead load as well as the cable shortening.
2) force due to tension from the various actions on the structure viz. other dead loads, superimposed load, live load, thermal etc.

The tension in stay due to other various actions as mentioned in (2) above on the constructed structure shall be arrived directly from the analysis of each action.

6.19 Loads during Construction.

The construction loads to be considered for design purposes shall be in line with the construction scheme of bridge. The effect of locked-in stresses due to erection loads shall be accounted for.

Checks shall be made to evaluate the effects of creep of the superstructure concrete on change of force in the stay cables and related changes in stresses in the superstructure during construction.

For design during construction stages, clauses under **Annexure A6** of IRC:112 shall be referred.

SECTION II: LOAD COMBINATIONS AND LOAD FACTORS

For the design of the elements of the cable stayed bridges (viz, Deck, Pylon, Pier Foundations, Stays) load combinations as per IRC:6 for Stability, Serviceability and Strength shall be followed. Additional combinations in respect of cable stayed bridges as given below shall be considered.

6.20 Combinations for SLS Check

Table: 6.1 Partial Safety factor for Verification of Serviceability Limit State for Stay Cables

Loads	Rare Comb 1	Rare Comb 2 (Congestion factor)	Rare Comb 3 (SV Loading)	Cable Replacement
Permanent load:				
Dead Load,	1.0	1.0	1.0	1.0
Creep and Shrinkage effects	1.0	1.0	1.0	1.0
Earth Pressure due to Back fill	1.0	1.0	1.0	1.0
Surfacing:				
Adding to effect of variable loads	1.2	1.2	1.2	1.2
d) Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Settlement Effects:				
Adding to Permanent loads	1.0	1.0	1.0	1.0
Opposing to Permanent loads	0	0	0	0
Variable Loads:				
Live Load		*		**
Leading Load	1.0	1.0	-	1.0
Accompanying Load	0.75	0.75	-	0.75
with Congestion factor	No	Yes	No	No
Special Vehicle Loading	-	-	1.0	-
Braking	1.0	-	-	1.0
Centrifugal	1.0	-	-	1.0
Wind Loads				
Leading Load	1.0	1.0	-	-
Accompanying Load	0.6	0.6	-	-
Live Load surcharge	0.8	0.8	-	-
Thermal Loads				
Leading Load	1.0	-	-	1.0
Accompanying Load	0.6	-	-	0.6
Cable Load	1.0	1.0	1.0	1.0

* Live Load without transverse eccentricity

** Reduced Live Loads Lanes

6.21 Combinations for ULS check

Table: 6.2 Partial Safety Factor for Additional verification of Stay Cable Strength

Loads	Basic combination	Basic combination (Congestion factor)	Basic combination (SV Loading)	Cable Break (Accidental)
Permanent load: Dead Load, SIDL except surfacing				
Adding to effect of variable loads	1.35	1.35	1.35	1.0
Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Surfacing:				
Adding to effect of variable loads	1.75	1.75	1.75	1.0
Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Earth Pressure due to Back fill:				
Leading Load	1.50	1.50	1.50	1.0
Accompanying Load	1.0	1.0	1.0	1.0
Variable Loads:				
a) **Live Load**				
b) Leading Load	1.5	1.5	-	0.75
c) Accompanying Load	1.15	1.15	-	-
d) Construction live load	1.35	1.35	-	-
with Congestion factor	No	Yes	No	No
e) Special Vehicle Loading	-	-	1.15	-
f) Braking	1.5/1.15	-	-	0.75
g) Centrifugal	1.5/1.15	-	-	0.75
Wind Loads				
Leading Load	1.50	1.50	-	-
Accompanying Load	0.9	0.9	-	-
Thermal Loads				
Leading Load	1.5	-	-	-
Accompanying Load	0.9	-	-	-
Live Load surcharge	1.2	-	-	-
Erection load	1.35	-	-	-
Cable force (at the end of construction stage)	1.35	1.35	1.35	1.35
Accidental Effects:				
Cable break	-	-	-	1.1

6.22 ULS check for Stay cables:

Cable forces under ULS loads shall be calculated as explained below.

1) Cable forces at the end of the stage construction which includes dead load and cable shortening shall be considered with factor 1.35.
2) Cable force due to other loads (for e.g. 1.5 LL for basic ULS combination above) shall be arrived from factored load applied on the completed structure.
3) Both the cable forces (1) and (2) above shall be combined to arrive at force F_{ed} in cable under ULS combination
4) Force F_{ed} obtained above shall be verified for ULS as per **Cl. No 7.8.2** in **Chapter 7**

6.23 For additional reading references [3] to [9] may be consulted.

CHAPTER - 7

ANALYSIS AND DESIGN

SECTION I – ANALYSIS

7.1 Introduction

The analysis of a cable stayed bridge is usually carried out using 3D frame method or finite element method by using various commercially available software or proprietary software developed by the design consulting firms. The capabilities of these software vary significantly and hence they are to be chosen judiciously depending on the intent of the analysis.

7.2 Structural Modelling

The models to be developed for the analysis also vary depending on the purpose of the analysis and stages of the design process, e.g. preliminary analysis for developing the concepts, feasibility, deciding the configuration & form of the bridge or detailed analysis for the final execution design.

Very often, different models need to be developed depending on type of loads e.g., seismic or wind loads. The model needs to be modified to carry out the construction stage analysis using the features available in the software. Sometimes, local analyses are required to be carried out in detail to investigate the local response of the structure, say at the anchorage or at other intricate locations.

Adoption of linear elastic static analysis or non-linear analysis (geometrical and/or material) shall be decided based on the size, configuration of the bridge and severity of the loads.

The choice of the type of elements to be used in the model, e.g., truss, beam, solid or shell etc., with their appropriate features available, is also an important decision which depends again on purpose of the analysis and the usage of the results of the analysis for the design.

Generally, simplified line element model of a bridge is adequate not only to understand the general behavior of the structure for the preliminary design, but also for seismic or aerodynamic analysis etc. A finite element model with solid and/or shell elements can be used for stress analysis for local parts of the structure, to have more insight. To trade off the computational efficiency and capability, one can establish a multi-element model to cater for the objectives as explained below.

7.2.1 *Line or Frame Element Model* ***(Fig. 7.1)***

In a line element model, components of a cable-supported bridge are modelled by line elements including beam elements, truss elements, and rigid links. Pylons and piers are usually modelled with beam elements based on their geometric properties. Cables are often modelled by either truss elements, or cable elements which take into consideration the geometric non-linearity due to cable sag; the static equilibrium profile of the bridge also needs to be calculated iteratively, based on the tension forces in the stay cables and the weight of the bridge deck.

The bridge deck, however, is more challenging to model. Usually, there are two approaches that can be employed to simplify the complicated bridge deck into a series of beam elements.

In the first approach, the deck is modelled as a central beam (spine beam). The equivalent cross-sectional area of the deck is calculated by summing up all cross-sectional areas of the deck.

In the case of a composite section, the areas should be converted to that of one single material, according to the modular ratio. The neutral axes and moments of inertia about the vertical and transverse axes are also determined. In a similar way, the mass moment of inertia of the deck should include those of all members, according to their distances from the centroid of the section.

Constraints, usually spring elements, rigid links or direct coupling of nodal displacements, are necessary to connect different parts of the model together and to enforce certain types of rigid-body features. For example, the nodes of the deck, bearings and towers do not coincide with each other. Rigid links are usually used to restrain their motions in different directions, depending on the bearing types and boundary conditions. Rigid links are also used to connect the spine beam with cables in the spine beam model.

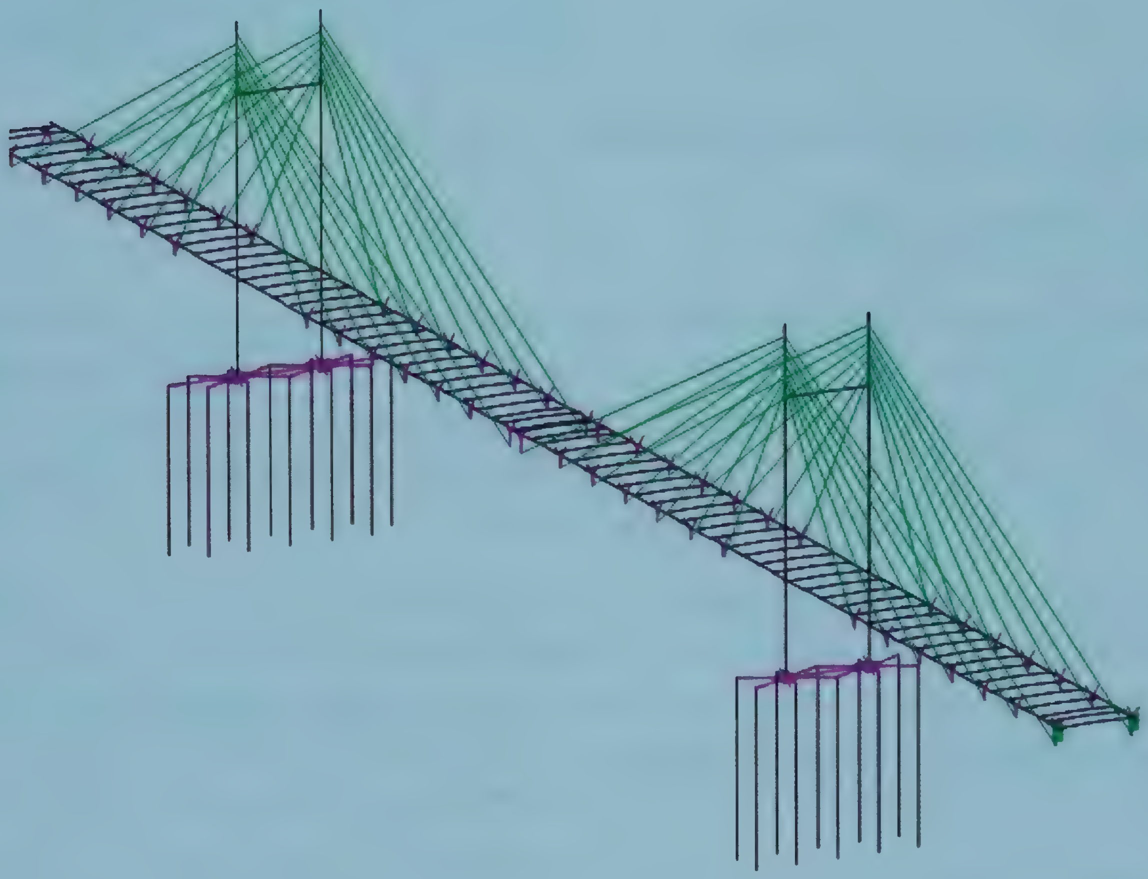


Fig. 7.1 3D Frame Model

7.2.2 *Multi-Element Model* ***(Fig. 7.2 & 7.3)***

To capture the local responses of some critical members and/or to obtain more accurate results, the entire bridge can be modelled with different types of elements. The ever-increasing capacity of computers makes this multi-element modelling more and more popular. In such a multi-element model, the components of interest can be modelled with shell elements or solid elements and other components with line elements. Special care must be taken at the interface between a solid element, a shell element and a beam element, because they have different DOFs. Multi-point constraints (MPC) are needed to connect the different elements in order to avoid displacement incompatibility among the nodes. Various MPC equations for beam to shell, shell to solid and beam to solid connections have been developed in terms of total energy principle and implemented in the commercial software.

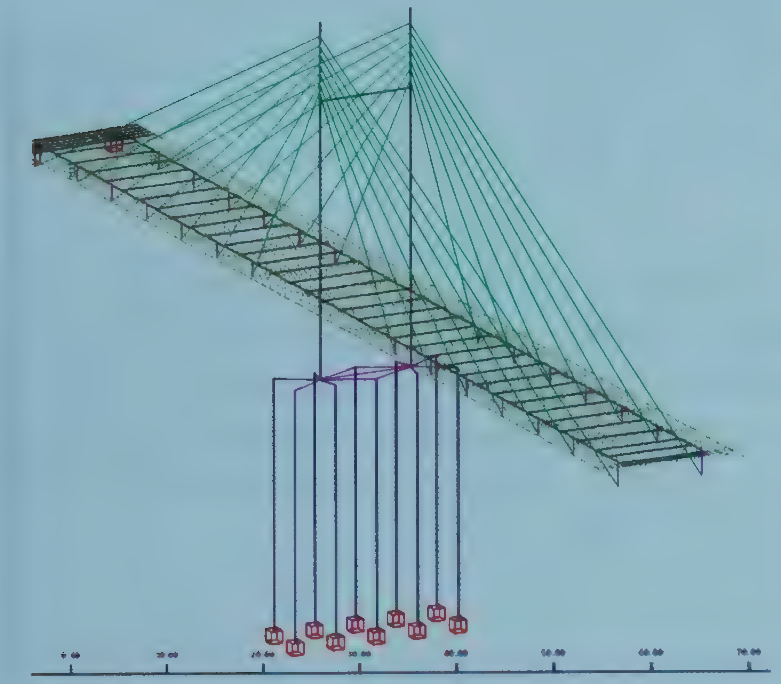


Fig. 7.2 Model with Line and Quad Element

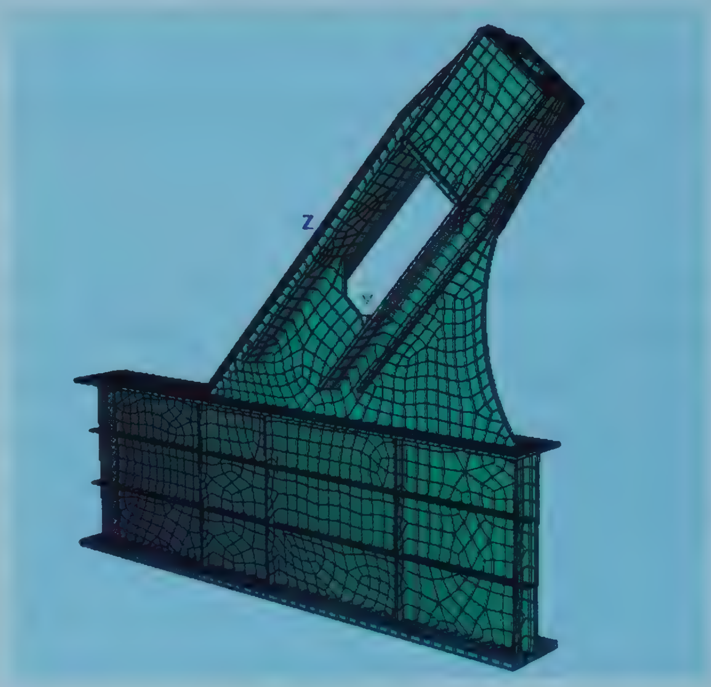


Fig. 7.3 FE Model of Anchorage

7.3 Geometric Nonlinearities

There are three main sources of geometrically nonlinear behaviour of cable-stayed bridges: the beam-column effect of the Deck as well as the Pylon, the large displacements (known as P-D) effect and the cable sag effect **[1]**. It is generally accepted that the latter is the most relevant of those and, therefore, its consideration is mandatory in cable-stayed bridge design, even for simplified models of such structures.

The Cable sag gives the most significant nonlinear effects and can be a decisive issue in the global behaviour of these structures. Cable modelling can be carried out using truss element (the modified elastic modulus element), cable element (the elastic catenary element) or multiple straight link discretisation. Element formulation shall include nonlinear solver to account for large displacement if the same is anticipated in case of longer cables.

Beam-column effect generally does not significantly contribute to the geometrically nonlinear behaviour of cable stayed bridges. Even though potentially important close to the ultimate load, neither moment-axial interaction nor bowing are important effects when compared with either cable sag or large displacement effect

For slender decks and pylon, Beam-column effect can be accounted either through the use of stability functions or by considering a narrow mesh refinement with a finite element solver accounting for large displacements.

7.4 Modelling Aspects

7.4.1 *Deck*

For the design of the structural elements in a cable stayed bridge it is sufficient in most cases to use a standard two- or three-dimensional line or frame analysis program.

For bridges with only one, central cable plane, a two-dimensional structural system is adequate to analyse the structure under vertical loading due to dead load and traffic load. This analysis gives the forces in the stay cables, and the axial forces, shear forces and bending moments in the girder and the pylons.

For the torsion induced in the girder under one-sided traffic load, the girder can be analyzed subsequently without taking the cable system and the pylon into account. The same type of analysis applies to the girder under lateral load, e.g. wind load.

In bridges with two cable planes the mathematical model has to be three-dimensional as depicted in **Fig. 7.4 (a)** i.e. with one beam element to model box girder and cable connected by rigid link element on both sides or **Fig. 7.4 (b)** i.e. with two longitudinal beam elements to model the main girders and a large number of transverse beam elements to model the transverse girders. The stay cables can in most cases be modelled as truss element carrying pure tension.

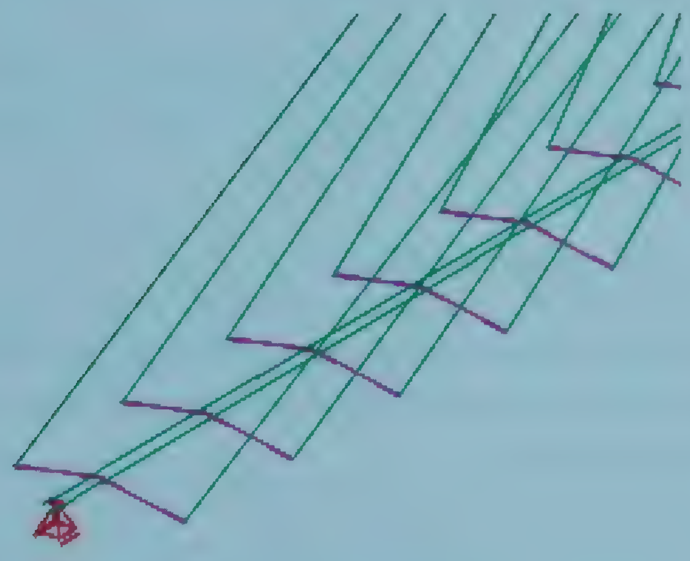


Fig. 7.4 (a) Analytical Model Twin Cable

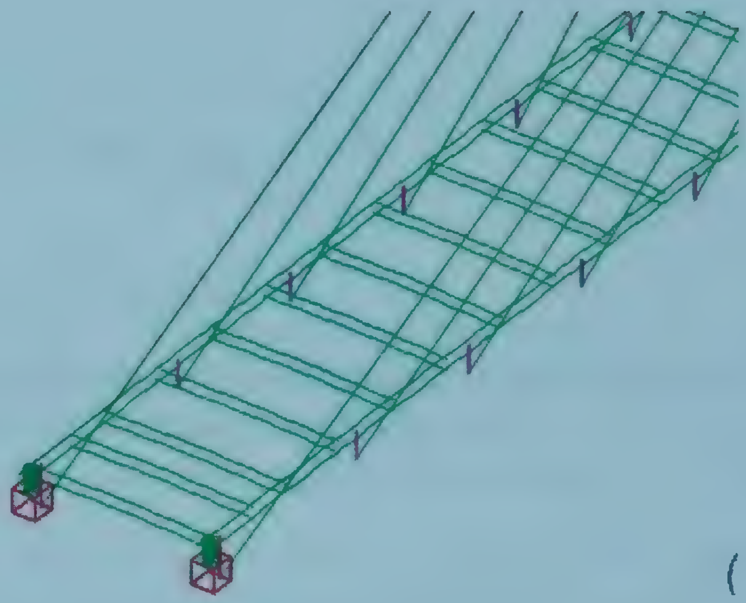


**Fig. 7.4 (b) Box Girder
Twin Cable with Plate Girder**

For bridges with a single box shaped girder and two cable planes, as illustrated in **Fig. 7.4 (a)**, the mathematical model should comprise a single longitudinal girder with the flexural and torsional stiffness of the box shaped main girder. At the points of cable attachment, the central beam should be joined to rigid link elements, as illustrated in **Fig. 7.4 (a)**.

7.4.2 *Cable and Stay Sags*

In the analysis, the stay cable is regarded as a straight member subjected to pure tension. In reality an inclined cable is always slightly curved due to the action of the own weight of the cable. For long stay cables, the sag tends to reduce the axial stiffness as the elongation is not only due to elastic strains in the cable wires but also due to reduction of the sag, as illustrated in **Fig. 7.5.**

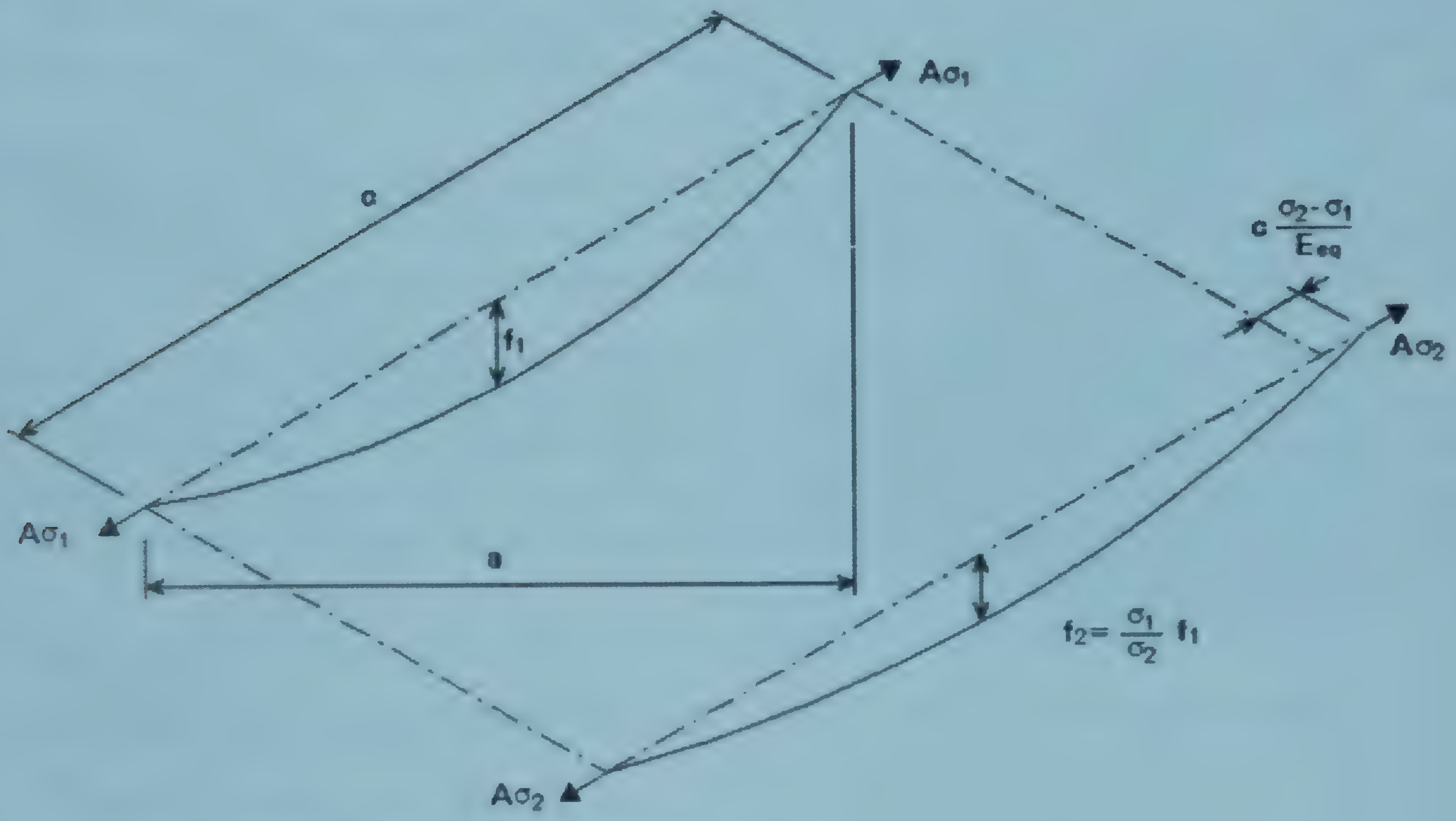


Fig. 7.5 Influence of Gravity on cable stiffness

For stay cables with horizontal projections up to 150 m and moderate stress variations, the sag effect can generally be disregarded, but for longer cables the stiffness of the cable support is overestimated if the axial strain is only considered.

The sag effect can be considered in the analysis by substituting the real modulus of elasticity E of the cable material by an equivalent modulus of elasticity E_{eq} determined by:

$$E_{eq} = \frac{E_{sc}}{1 + \frac{\gamma^2 a^2}{24}\left(\frac{\sigma_1 + \sigma_2}{\sigma_1^2 . \sigma_2^2}\right) E_{sc}}$$

(.....Eq.1)

where,

γ - is the density of the cable material

a - the horizontal projection of the cable

σ_1 - is the initial dead load, stress

σ_2 - the final stress (from dead + live load)

E_{sc} - Modulus of Elasticity for stay cable material

Equation (1) which introduces a secant modulus, requires an iteration in the structural analysis, as the final stress σ_2 is unknown from the beginning.

For stay cables with moderate stress variations and horizontal projections up to 250 - 300 m, it is allowable to substitute the secant modulus of Equation (1) by a tangent modulus E_{tan} derived simply by inserting $\sigma_2 = \sigma_1$ in Equation (1):

$$E_{tan} = \frac{E_{sc}}{1 + \frac{\gamma^2 a^2 E_{sc}}{12\sigma_1^3}}$$

(.....Eq.2)

In the latter case an iteration can be avoided as the tangent modulus depends only on the initial stress σ_1.

When applying the more exact secant modulus approach, the first step in the iteration is often based on axial cable stiffnesses found from Equation (2), whereas the following steps are based on stiffnesses determined from Equation (1).

Presently, several commercial software are available which can calculate cable sag and effective stiffness of the stay cables during construction and service.

7.4.3 *Pylon*

The pylon is subjected primarily to compression from the vertical components of the cable forces, and can be modelled as line element. Second order effects in the pylon shall be considered in the design based on the **Section 7.5.3** of this Guideline.

7.5 Structural Analysis

7.5.1 *Linear and Pseudo-Linear Analysis*

The linear analysis of the numerical model of cable-stayed bridge uses the standard beam and truss elements. Cable-stressing forces are applied as loads, by evaluating the shortening in length of each cable element.

Pseudo-linear analysis is a linear iterative method in which only the modified elastic modulus of the truss elements, used to model the stay-cables, is updated according to the current cable tension, to account for the cable sag effect. No geometry updating is made and so, as in linear analysis, out-of-balance forces are not evaluated. The relevant advantage of this approach is that it may be undertaken with common linear analysis software.

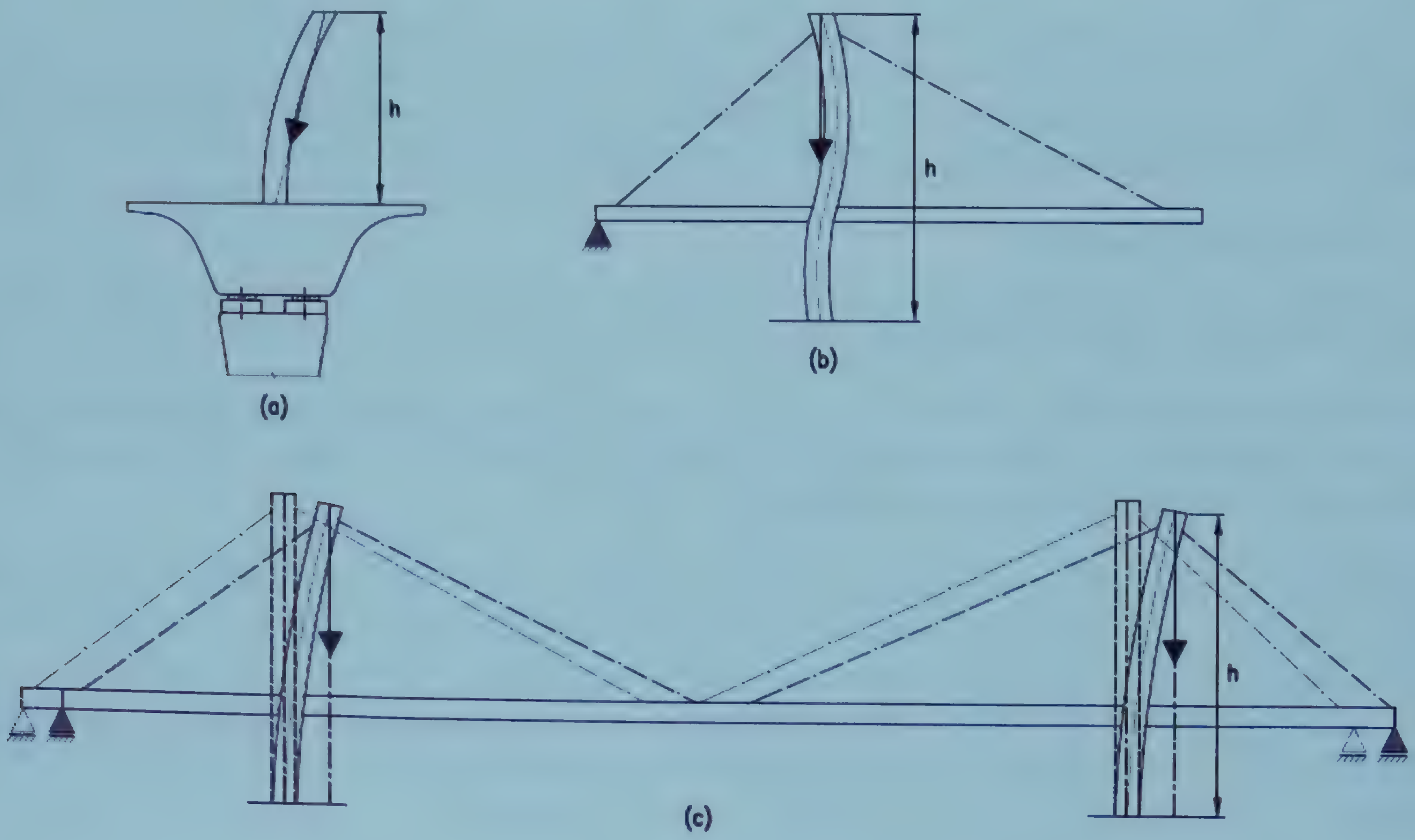
7.5.2 *Nonlinear Analysis*

Eurocode 1992-2 provided methodology for Non-linear analysis of concrete bridges, which accounts for the non-linear nature of the material properties. This includes the effects of cracking in concrete and non-linearity in the material stress–strain curves. The analysis then ensures both equilibrium and compatibility of deflections using these material properties. Such an analysis may be used at SLS and ULS. Non-linear analysis also models non-linearity in structural response due to the changing geometry caused by deflections. This is important in the design of elements where second-order effects are significant, as explained further in this guideline. It is for second-order calculation for slender members, such as bridge piers and pylon, that non-linear analysis is often particularly beneficial, as the simplified alternatives are usually quite conservative.

7.5.3 *Second Order Effects*

Second-order effects are additional action effects caused by the interaction of axial forces and deflections under load. First-order deflections lead to additional moments caused by the axial forces and these in turn lead to further increases in deflection. Such effects are also sometimes called P-Δ effects because additional moments are generated by the product of the axial force in the element and system deflections. IRC:112 provides simplified method and general method for obtaining second order effects.

The nominal curvature method can be used to obtain second order effects in pylon when the pylons have simple forms as shown in **Fig. 7.6**, where three examples are given to arrive at effective length of the pylon.



(a)

(b)

(c)

Fig. 7.6 Stability of Pylon

Fig. 7.6 (a) shows the lateral buckling of a free-standing pylon supporting a pure fan system at the top. The column effective length 1_c is equal to the pylon height 'h' due to the fact that the cable plane rotates with the anchor point at the pylon and the resultant of the stay cable forces still points towards the bridge axis. If this effect is neglected, then the effective length of the free-standing pylon would be twice the pylon height, i.e. a much more on the conservative side.

Fig. 7.6 (b) shows the buckling of a pylon in the longitudinal direction in a system where the girder is longitudinally supported at the end pier. In this case, the anchor cable leading from the fixed bearing to the pylon top restrains the pylon top in the longitudinal direction and the column effective length in the plane is therefore close to 0.7 times the pylon height.

Finally, **Fig. 7.6 (c)** shows the buckling of a pylon in a system where the girder has no longitudinal restraint through fixed bearings. In this case, the buckling is accompanied by a longitudinal sway of the girder. The column effective length is, consequently, twice the pylon height.

General method based on non-linear analysis as per IRC:112, can also be used to obtain second order effects in case of slender pylon.

When second-order calculations are performed, it is important that stiffnesses are accurately determined. It is therefore required that the analysis considers the effects of cracking, non-linear material properties and creep. This can be done either through a materially non-linear analysis or by using linear material properties based on a reduced secant stiffness as per IRC:112. Imperfections must be included as described in IRC:112 as these lead to additional first-order moments and consequently additional second-order moments in the presence of axial compression. Soil–structure interaction must also be considered.

7.5.4 *Method of Analysis for Specified Limit States*

Structural analysis shall be carried out by an appropriate model and analytical theory for specific limit states. The structural analysis shall be generally done with analytical models that can evaluate the stress produced by load acting on the structure. In addition, the analysis should consider the effects of creep and shrinkage.

a) Structural Analysis Method for Limit State of Serviceability

Calculation of member force to be utilized for examination under serviceability limit state shall be based on the Pseudo linear analysis as explained above. Usually, the stiffness to be utilized for calculation of member force may be estimated under the condition that the entire cross-section area is effective.

In regard to the estimation of displacements and deformations for structure, the concrete structure shall be generally regarded as an elastic body, and decrement of stiffness due to occurrence of cracks and effects of shrinkage and creep shall generally be taken into account.

b) Structural Analysis Method for Ultimate Limit State

In regard to the estimation of member force to be utilized for examinations of section failure under ultimate limit state, Pseudo linear analysis may be used in general. In case of the slender pylon or deck, results as per the above analysis may need to be verified with general method of second order effects based on nonlinear analysis, as per IRC:112.

For dynamic analysis, reference shall be made to **Chapter 8** of this guideline.

c) Structural Analysis Method for Fatigue Limit State

An estimation of member force to be utilized for examinations of fatigue limit state shall be based on the Pseudo linear analysis.

7.6 Initial Shape and Determination of Cable Forces

The cable stayed bridge consists of three principal components, namely deck/girders, towers and inclined cable stays. The deck system is supported elastically at points along its length by inclined cables so that it can span a much longer distance without intermediate piers. The dead load and traffic load on the deck system are transmitted to towers by inclined cables. High tension forces exist in cable stay which induce the compression forces in tower and parts of the deck system. Since high pretention force exists in inclined cables before live loads are applied the initial geometry and the prestress of cable stays are dependent on each other. They cannot be specified independently as for conventional steel and reinforced concrete bridges. Therefore, the initial shape, i.e., the geometric configuration and prestress distribution of cable stayed bridges, must be determined prior to analysing them.

In the design of these bridges, the assessment of initial cable forces and the procedure of stay stress adjustments during erection is very important, to respect the requested geometric profiles of deck and towers at the end of construction. At the end of erection, the girder longitudinal profile must satisfy aesthetic and functional requirements, possibly presenting a convenient pre-camber; the towers must keep the vertical profile, in order to avoid second order effects and to satisfy architectural demands.

Several techniques for the detection of initial stress distribution of pretension exist:

i) Simply Supported Beam Method

Assuming that each stayed cable supports approximately the weight of one segment, corresponding to the longitudinal distance between two stays, the cable forces can be estimated conveniently. It is necessary to take into account the application of other loads (wearing surface, curbs, fences, etc.). Also, the cable is placed in such a way that the new girder element is positioned correctly, with a view to having the required profile when construction is finished.

ii) Method of Continuous Beam on Rigid Supports

By assuming that under the dead load the main girder behaves like a continuous beam and the inclined stay cables provide rigid supports for the girder, the vertical component of the forces in stay cables are equal to the support reactions calculated on this basis. The tension in the anchorage cables make it possible to design the pylons in such a way that they are not subjected to large bending moments when the dead loads are applied.

iii) Optimization Method

In the optimization method of determining the stresses of the stay cables under permanent loads, the criteria (objective functions) are chosen so the material used in girders and pylons is minimized. When the internal forces, mainly the bending moments, are evenly distributed and small, the quantity of material reaches a minimum value. Also, the stresses in the structure and the deflections of the deck are limited to prescribed tolerances

iv) Shape iteration method

A finite element procedure for determining the initial shape of cable-stayed bridges under the action of dead loads and pretension in inclined cables is presented e.g., in Wang **[12]**. The cable-stayed bridges are analysed with all the geometrical nonlinearities with the available software. Based on a reference configuration and an assumed cable pretension force, the equilibrium configuration under dead load is then found. Further, by adjusting cable forces, a 'shape iteration' is carried out and a new equilibrium configuration, i.e., a more reasonable initial shape, can be determined. The shape iteration is then repeated until the desired tolerance is achieved.

7.7 Construction Stage Analysis and Adjustment of the Cable Forces

During the construction or service stage, many factors may induce errors in the cable forces and elevation of the girder, such as the operational errors in tensioning stays or the errors of elevation in laying forms. Further, the discrepancies of parameter values between design and reality such as the modules of elasticity, the mass density of concrete, the weight of girder segments may give rise to disagreements between the real structural response and the theoretical prediction. If the structure is not adjusted to reduce the errors during construction, they may accumulate, and the structure may deviate away from the intended design aim. Moreover, if the errors are greater than the allowable limits, they may give rise to the unfavourable effects to the structure. Through cable force adjustment, the construction errors can be eliminated or reduced to an allowable tolerance. Additionally in the service stage, because of concrete creep effects, cable force may need to be adjusted; thus, an optimal structural state can be reached or recovered.

For constructing the bridge and predicting the structural behaviour of the bridge during its erection procedure, the geometric configuration and forces in members of the bridge structure at each erection stage have to be known and examined in detail.

To reach the design aim, an effective and efficient simulation of the construction process step by step is very necessary. The objectives of the simulation analysis are

1. To determine the forces required in cable stays at each construction stage.
2. To set the precamber of the girder segment.
3. To find the consequent deformation of the structure at each construction stage.
4. To check the stresses in the girder and pylon sections

Construction stage analysis technique can be forward step analysis or backward step analysis

i) Forward Step Analysis

The forward process analysis of cable-stayed bridges during construction is performed by following the actual sequence of erection stages in bridge construction. Creep effects can be considered; however this method has limitation that actual lengths of the cables (which make the final elevation of the deck) are not known initially and hence requires some trials. But due to availability of the specialised commercial softwares to carry out this analysis trials and limitations of this method can be taken care.

ii) Backward Step Analysis

The backward process analysis of cable-stayed bridges during construction is carried out by following the direction reversed to the sequence of erection stages in the bridge construction. Backward step analysis, which starts with the final aim of the structural state and disassembles segment by segment in a reverse way. The disadvantage of this method is that the creep effects may not be able to be defined. However, values obtained from the assemblage process may be used in this analysis.

SECTION II – DESIGN

7.8 Design of Cables

The design of cables for cable stayed must comply with requirements of this section. This section of the guideline is generally in line with **Fib 89 [1].**

7.8.1 *Cable Properties*

Guaranteed ultimate tensile strength

For parallel-Strand or Parallel-wire stay cables the Guaranteed ultimate tensile strength of the cable F_{GUTS} is given by

$F_{GUTS} = f_{GUTS} \, . \, n \, . \, A_p$

Where,

n is number of strand / wires

A_p Area of each strand / wire

f_{GUTS} Guaranteed ultimate tensile stress of the cable

7.8.2 *Ultimate Limit State for Cables*

The tension of the cable under ULS load combinations (F_{ed}) during the construction phase and operation must satisfy following

$$F_{ed} \leq \frac{F_{GUTS}}{\gamma_m}$$

γ_m - 1.35, Partial Safety factor for Cable material

All Anchorage parts (Anchor Blocks, Socket case, Jaws, Swaged Sleeves etc) and the adjustment parts between cable anchorage and the structure should be designed to be stronger than ultimate strength F_{GUTS} of the cable they are to anchor.

In addition, under seismic or Accidental loads, there must be no irreversible plastic deformations on the parts which might hinder cable stay removal.

The parts of the structure into which the cable stay is connected, and, which transfer cable tension to the structure, shall be designed for Limit State condition as per the IRC and must be able to take force of 1.0 F_{GUTS} under ULS combinations.

7.8.3 *Seismic Design for Stay Cables*

All cables need to be designed for earthquakes, e.g., to avoid plastic deformations in the cable (values to be limited as per **Section 7.8.2**) considering flexural effects under large deformations of the structure.

Time history analysis as per **chapter 8** should be performed to verify seismic demand and differential dynamic force conditions for any use of saddles in seismic zone IV and zone V. When seismic time-history analysis predicts net cable tension loads less than 10% of F_{GUTS}, cables and their anchorages shall be designed to assure adequate seismic performance (e.g. wedges are secured to cable in place). In addition, verification testing of cable lift off at the anchorage system and saddle shall be proposed by the cable supplier and approved by the designer on a project-by-project basis.

Saddles that depend solely on friction, if used in seismic zone IV and zone V, should be checked with detailed structural investigation of the bridge, such as time history analysis, using relevant site-specific ground motion input and friction properties of the saddle.

7.8.4 *Fatigue Limit State*

A) Stay Cable and Anchorages

The fatigue damage usually occurs at the anchorages, because of varying axial load and repeated bending caused by angular deviation at the ends. These angular deviations are principally due to cable stay vibrations caused by wind and due to rotation of the bridge deck.

- When using cable systems designed and tested for angular deviation during fatigue test for fatigue angular deviations and service angular deviations as per the recommendations in **Chapter 10**, fatigue angular deviation need not be considered in design.

- Damping of the cable vibrations by means of dampers, stabilizing stay cable textured stay pipe shall be done if required. In addition to Internal Dampers at stay anchorage's location, provision of external dampers shall be also explored, if cable vibration requirements are not met with.

- Hinging of the cable anchorage relative to the structure during the construction phase (for joint), or use of an effective procedure for adjusting the axial orientation of anchorages to avoid unforeseen permanent angular deviation between the cable stay and the deck or pylon can be adopted.

Further with above detailing care taken into consideration, the stay cables fatigue strength is guaranteed if following two conditions are met:

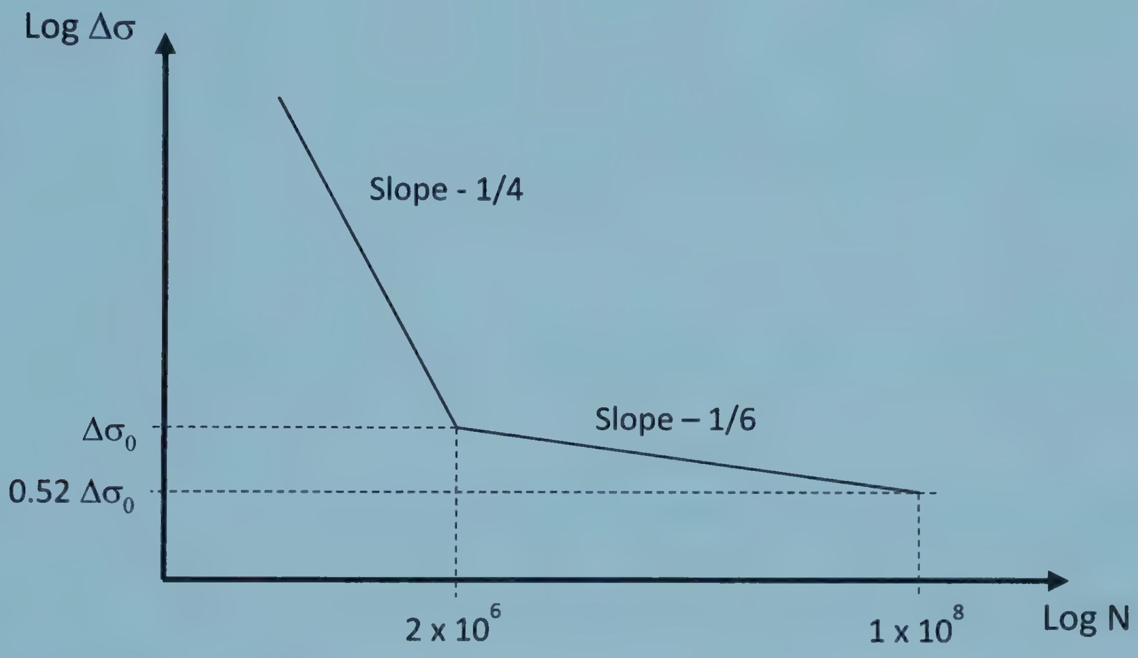
1) Qualification of cable system in accordance with **Chapter 10**

2) Verification of the cable system for variable stresses in cable (demand) compared to design strength of the fatigue stress range.

Annexure A2 of this guideline explains different approaches adopted internationally for fatigue design of stay cables. The fatigue design of stay cable to be considered for the cable stayed bridges as per the provisions mentioned below:

B) Design S-N fatigue P.C. Curve for Stay Cable

The shape of the S-N Curve (Wohlers Curve) of a cable stay depends on the cable stay system. It has a bilinear shape and changes direction at 2 millions cycles for the stress range of $\Delta\sigma_0$



In the absence of additional data, it is acceptable to take the fatigue strength at 2 million cycles $\Delta\sigma_0$, of a cable stay being the same as the stress range $\Delta\sigma_{test}$ applied during the mechanical qualification test. **Chapter 10** recommends a $\Delta\sigma_{test}$ range of 200 MPa for cable stayed bridges with upper stress of 0.45 f_{GUTS}.

As an approximation, the slope of the second part of the curve can be estimates to be 1/6. The endurance limit of the cable at 100 million cycles is therefore:

$$\Delta\sigma_0 \left(\frac{10^8}{2\ \times 10^6}\right)^{-\frac{1}{6}} = 0.52 \times \Delta\sigma_0$$

C) Fatigue Verification

The cable stayed bridge for highway loading shall be designed to withstand 100 million crossings of the Fatigue Vehicle and it should therefore be checked that

$$\Delta\sigma_{fat} \leq \frac{0.52.\Delta\sigma_{test}}{\gamma_{Mf}}$$

$\Delta\sigma_{fat}$ – stress range due to fatigue vehicle as per IRC:6

γ_{Mf} – Partial Safety Factor – 1.25

The stress range $\Delta\sigma_{fat}$ due to fatigue vehicle shall be arrived at by calculating absolute difference in maximum and minimum stresses in cable due to passage of Fatigue Vehicle and multiplying by factor 1.4 to account for longer spans and uncertainty in load.

D) For other elements and details

Fatigue check for other elements of bridge for reinforcement, structural steel relevant codes of IRC shall be referred.

7.8.5 *Serviceability Limit State for Cables*

i) During Service

The Stay cables shall be checked for maximum stresses in rare combination.

$f_{sis} = 0.5\ f_{GUTS}$

f_{sis} - maximum permissible stresses under rare SLS combinations

f_{GUTS} - Guaranteed Ultimate tensile strength of the stay cable.

ii) SLS check under Construction and cable replacement

The permissible axial stresses during construction and cable replacement under rare SLS load combinations shall be as below

$f_{sis} = 0.6\ f_{GUTS}$

7.9 For additional reading references [2] to [12] may be consulted

CHAPTER - 8

SEISMIC EFFECTS

8.1 Introduction

The Chapter includes Guidelines on Seismic Effects bassed on State of the art in Earthquake Engineering including Spatial Variablity Effects of Ground Motion and Seismic Protection Devices for Structural Response Control. Cable stayed bridges are long span and flexible structures, typically having long vibration periods, low damping, complex vibration modes, and usually subject to spatial variation of ground motion at different pylons/supports. The long fundamental period may result in low design seismic forces governed by the minimum design forces prescribed by **Clause 5.4** of IRC:SP:114. On the other hand, these bridges undergo large seismic deformations, needing consideration of second-order effects in their structural analysis and design. Supplemental energy dissipation may be useful in controlling the displacement. Due to coupling between the longitudinal, transverse, vertical and torsional modes of vibration, three-dimensional modeling and dynamic analysis considering adequate number of modes is necessary. The seismic design of these bridges shall be governed by IRC:SP:114. However, due to long spans and complex dynamic behavior, some additional considerations are required for seismic design of cable stayed bridges. The peculiarities of seismic behavior of cable stayed bridges and additional requirements for seismic design of these bridges, are discussed in this chapter.

8.2 Seismic Conceptual Design

The conceptual seismic design is an important step of the total design process. Seismic performance is dependent on the conceptual design, particularly the structural form of the cable stayed bridge, tower type, and cable layout. In the conceptual design phase, the structural form should be selected based on design considerations for static loads, seismic resistance and aerodynamic stability. To decide the structural form, cable layout and tower types, pylons and foundations, it may be desirable to carry out a parametric study amongst few chosen structural forms, tower types and cable layouts, to determine their impact on seismic behavior using the quick response spectrum method. Seismic conceptual design may require considerations of four design aspects which produce minimum seismic forces and seismic displacements: deck type, deck-tower connection, arrangement of side piers, and deck-side pier transverse connection. Reference may be made to **Chapter 4** and Calvi et. al. **[3]** for preliminary design recommendations. In general, for a cable stayed bridge with several auxiliary piers, it is beneficial for deformation and forces induced in the structure that all the side piers offer transverse restraint. Similarly, a full floating (without any

deck-pier connection) configuration with fuses and supplemental energy dissipation **Fig. 8.1** can result in reduced forces as well as controlled displacement. Under eccentric gravity loading and wind forces, the rigid fuses transfer the deck forces to the pylons, whereas under severe seismic loading the fuses yield, thus activating the dampers, resulting in energy dissipation.

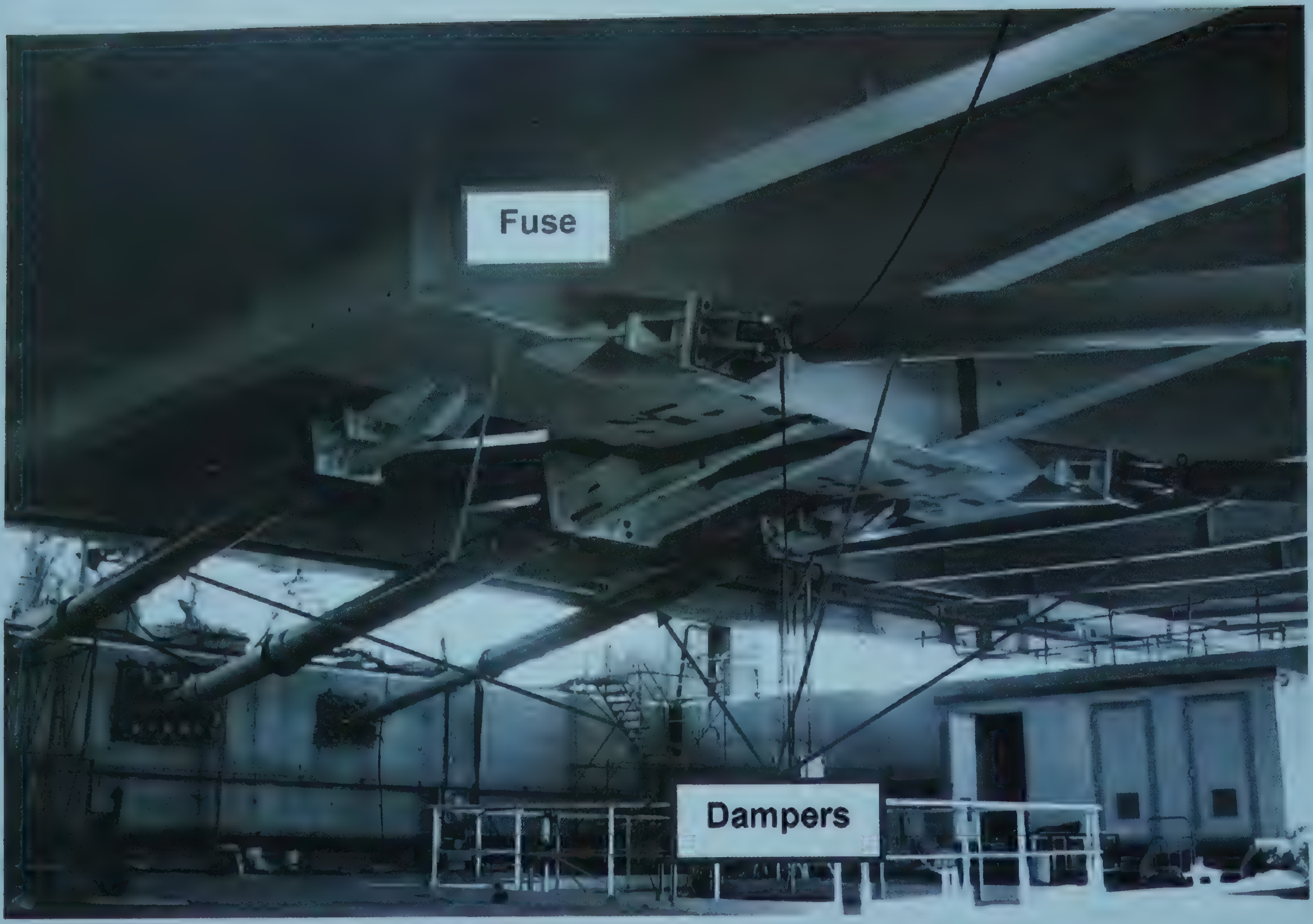


Fig. 8.1 Fuses and Supplemental Damping used for Fully Floating Superstructure of a Cable Stayed bridge [1]

8.3 Seismic Considerations

Cable stayed bridges are composed of various types of components with different stiffness and damping characteristics. These result in coupled modes and hence require some special considerations during their seismic design. Following are the main points to be considered in seismic design of cable stayed bridges:

i) These bridges usually do not have clearly distinguishable modes of vibrations in longitudinal, transverse, and vertical direction, as in the case of simple bridges and buildings. Due to coupling of modes, seismic action in one direction causes forces in other direction. The pylons are subjected to axial force and bi-directional bending actions due to longitudinal, transverse and vertical components of earthquake, along with gravity loads. Therefore, the cable stayed bridges should be designed for combined effect of the two horizontal and vertical components of earthquake. The 100:30:30 combination rule **(Clause 4.2.2 of IRC:SP:114)** can be used for combining the member actions obtained from response spectrum analysis. In case of time history analysis, all the three components of ground motion need to be considered and the effect is automatically combined in appropriate proportion.

ii) The deck-pier connection and expansion joints may get damaged when the movements of bridge component exceed their capacity (refer **Chapter 11**); these bridges may need large expansion joints to cater for large movements in earthquakes. At design basis earthquake (DBE), the damage to expansion joint should not hamper movement of vehicles. Use of dampers and fuses may be considered for reducing the excessive movement.

iii) Long span cable stayed bridges are sensitive to spatial variation of ground motion, because of wave passage effect and variation in geotechnical conditions below the tower supports. The spatial variation of ground motion results in large pseudo-static forces in addition to the dynamic forces. Large forces and displacements may be induced in the structure because of these factors. Therefore, multi-support response spectrum (MSRS) or multi-support ground motion analysis should be performed in case of bridges having distance between farthest supports interconnected by a continuous superstructure, more than the limit given in **Section 8.5**. The consideration of spatial variation is even more important in case of bridges having piers/towers located on different types of soil strata.

iv) Cable structures depict geometric nonlinearity. Due to sag in the cable, it shows a nonlinear force-displacement behavior, as part of the applied force causes straightening of cable, without any strain. **Chapter 7** provides different methods to take into account the geometric nonlinearity, including that due to cable sagging. Further, the cables cannot take any compressive force and therefore, it should be ensured that the cables are in tension during the whole time-history of the earthquake. The pre-tensioned cables of bridges may come in compression due to vertical vibrations of the deck caused by combined effect of horizontal and vertical ground motions; their status of tension should be checked under seismic conditions.

v) Long cables may have their own period of vibration close to the period of vibration of the pylon or superstructure. This may lead to swapping of energy between these components. This phenomenon is called cable-bridge interaction and must be considered in seismic analysis of cable stayed bridges. This can be easily considered by dividing the cables into smaller elements and lumping the masses at intermediate nodes.

vi) The long spans and tall pylons cause large overturning moments at the base requiring large size foundations. Also, the foundation of the bridge may destabilize due to soil liquefaction. The foundations shall be modelled considering the nonlinear behavior of soil as explained in **Section 8.4.5**. Further, the foundation should be designed for capacity forces being transmitted by the pylon, including overstrength, according to **Clause. 7.8 of IRC:SP:114**.

vii) High strength cables used to support the cable-stayed superstructure have very low ductility and should be designed to remain elastic, even during the most severe earthquake.

8.4 Modelling and Analysis for Seismic Design

8.4.1 3D Modelling of Structure

Cable stayed bridges undergo complex and coupled modes of vibration under earthquakes. The torsion of superstructure is combined with transverse vibrations. Similarly, the pylons are also subjected to torsion as a result of horizontal and vertical vibrations of the superstructure. Therefore, it is important that a 3-D model is used for seismic analysis and all the three components of earthquake motion are considered to act simultaneously. A single pier or single spine model of superstructure is not adequate to consider the torsional effects. The 3D model of a cable stayed bridge can be developed using line or frame elements or multiple types elements, as per **Section 7.2** of these guidelines. If a single spine model of a cable stayed bridge having a single plane of cables along the centerline of the super-structure, is used, the torsional inertia of the superstructure and spatial location of the bearings with respect to centerline of the superstructure need to be modelled adequately. This can be done by using rigid appendages projecting from the spine and connecting the bearings with the spine.

8.4.2 *Adequate Mode Participation*

The dynamic characteristics (i.e. free vibration modes) of a structure are classified according to their action direction: longitudinal, transverse, vertical and torsional. The natural vibration modes can occur in each of these directions and there can be coupling between the modes. The vibration modes of cable stayed bridge are complex. The initial modes of vibration show long periods, of the order of several seconds, and these are fundamentally deck modes. These are followed by cable vibration modes, coupled with the deck. The tower vibrations are usually higher-order vibration modes, which can be coupled with the deck depending on support conditions. The modes are difficult to separate when they are highly coupled. For typical cable stayed bridges, there are strongly coupled modes like bending and torsion in the three orthogonal directions. The vertical vibrations of deck, induced due to vertical ground motions at the tower supports, could be quite important; so also, the torsional motion of the bridge induced due to two simultaneous components of horizontal earthquake motions at tower supports.

As the seismic response in case of cable stayed bridges, is contributed by a large number of coupled modes, it is necessary to perform dynamic analysis for seismic design of cable stayed bridges. The equivalent static method which is essentially a single mode method is not adequate for these bridges. The dynamic analysis can be performed either by modal analysis method using response spectrum, or a modal response history analysis using appropriate ground motions, or a direct time history analysis. Naturally, the direct time history analysis is the most accurate being a rigorous analysis procedure, which inherently considers the contribution of all the modes of vibration. However, this may be a time-consuming method and requires skill in selection of appropriate ground motions. Therefore, it is recommended to use a modal analysis for preliminary design of cable stayed bridges, and the direct time history analysis for design verification at final stage of seismic design.

While using the mode superposition method, it is important that enough modes are considered to result in 90% mass participation in all the directions. Sometimes it is difficult to get 90% mass participation in each direction of the cable stayed bridge. In such a case, Ritz vectors may be tried, otherwise it may be required to perform a direct time history analysis, considering three components of ground motion.

8.4.3 *Design Hazard Level, Importance Factor and Response Reduction Factor*

Euro code suggests that "for bridges where the seismic response may be dominated by higher mode effects (e.g. cable-stayed bridges), or where the detailing of plastic hinges for ductility may not be reliable (e.g. due to a high axial force or a low shear-span ratio), a behaviour factor of q=1 is recommended, corresponding to elastic behaviour." Accordingly, the cable stayed bridges shall be designed for the DBE level earthquake forces with response reduction factor, R equal to 1.0 and the importance factor, I =1.5. The earthquake action in both the horizontal and vertical directions shall be considered.

8.4.4 *Damping*

The damping in Cable Stayed Bridges is characteristically low, smaller than 5% of critical damping, normally assumed in the bridges **[2,3,4].** The total damping in the bridge depends on the damping in individual components of the bridge; deck, towers, cable system, pylons, foundation, and founding soil. The damping in each of the component members is different. The dissipation mechanism in the seismic response of cable stayed bridge comprises of structural damping, bearing friction, internal slip of wires inside the cables (friction), foundation radiation, aerodynamic damping and system damping, due to interaction between the deck, cable system and the tower. To carry out seismic analysis, the equivalent damping can be worked out based on weighted damping proportional to the strain energy stored in each component. The estimated value of damping is also sensitive to the relative participation of modes. The damping reduces for higher modes and with increasing span. The damping matrix required for direct step by step integration method can be formed using Rayleigh damping. This requires specification of the damping in two modes of the vibration. The damping in Active or Passive devices can be included in the modeling of the device in the mathematical model in the form of hysteretic damping.

In a simplified case, the material of the deck can be used to decide the damping factor. In absence of a better estimate of damping, 2%, 3%, and 4% of critical damping can be considered for cable stayed bridges with steel, steel-concrete composite and RC decks, respectively. For hybrid decks, the lowest value applicable to main span will be used. For construction stage pylon material to be used to select the appropriate amount of damping. In direct time history analysis, the 2%, 3%, and 4% Rayleigh damping may be considered corresponding to fundamental mode and the mode resulting in 90% mass participation.

8.4.5 *Consideration of Effects of Soil*

In general, soil below the founding level has a complex interaction with the structures during earthquake. To consider the Complete soil-structure interaction, soil is modelled as continuum and seismic wave propagation through the soil is also considered. The soil flexibility, wave propagation, kinematic interaction, inertial interaction, and radiation damping, all the effects of SSI are considered together in this approach. This is the most accurate method of assessing SSI, but computationally intensive. Further, this requires extensive field and lab investigation of the founding soil, preferably up to the bed rock level. The soil properties (stiffness, strength and hysteresis parameters) estimated through the investigations are required for nonlinear continuum material modelling of the soil. Further, the free field ground motions selected at the surface level should be de-convoluted at level of the base of the FE model considered. As soil is a semi-infinite medium and only a finite domain is considered in the finite element model, free-field or absorbing boundary conditions are required to be applied at the domain boundaries. Many of the available computer software provide these modelling options.

In the soil-structure interaction, two effects of soil are the most important: (i) soil amplification of ground motion due to impedance contrast in the medium of wave propagation, and (ii) soil flexibility resulting in elongation of period and permanent deformation of the structure. In the decoupled method of SSI, these two effects are considered independently. In this method, the soil may be modelled using nonlinear (bi-linear or multilinear) springs and the wave propagation through the soil is considered separately, using 1D wave propagation. Several public domain software (e.g. SHAKE and DEEPSOIL) are available for 1D wave propagation. These can consider the soil as equivalent linear or nonlinear layered medium. The soil properties for each layer need to be specified. The de-convoluted ground motion is applied at the base of the soil column and the seismic motion at different levels in soil is obtained using the 1-D wave propagation. The ground motion obtained at different levels in the soil is applied at the corresponding springs. It is a reasonably accurate and efficient method, most commonly used internationally for practical SSI problems. The American Petroleum Institute (API) and other standards recommend this procedure and provide guidelines for developing nonlinear/bilinear soil-spring properties. Based on API and other guidelines a separate document, IRC B-3.7 **Annexure-F [5],** has been developed by IRC for estimating the modelling parameters of soil springs in different types of soils, including liquefiable soils. This document can be used to develop the soil spring models for pile and well foundations. Shallow foundations are less probable in case of cable stayed bridges, but such equivalent spring models are available in literature for shallow foundations also.

A further simplified procedure to consider the major two effects of SSI is possible, in which the soil amplification is considered using the code-based response spectra/amplification factors or using the site-specific earthquake design parameters, whereas the soil flexibility effects are considered using nonlinear/bilinear equivalent springs. However, a nonlinear time-history analysis is needed in this case also, which can be handled by most of the software available and used in Indian industry now-a-days.

For quick analysis for preliminary design or if the designer does not want to go for the nonlinear analysis there is an approximate method to use this modelling in linear analysis software, by performing iterations. In this iterative equivalent linear analysis, first a probable displacement of foundation is estimated using the fixed-base design forces or any other appropriate assumption. The secant stiffness corresponding to estimated displacement is computed using the bilinear force-displacement curves for soil spring. Then, using the secant stiffness as the equivalent linear stiffness, analysis performed using any of the commonly used software, and the resultant foundation displacement is computed. If it is close to the originally assumed displacement, the solution is accepted otherwise further iterations are performed using a new assumed foundation displacement. The advantage of this iterative equivalent linear procedure is that it can be used with response spectrum method also and provides reasonably accurate results for practical design. However, in case of important cable stayed bridges, it is desirable to make a final verification of the design using nonlinear time history analysis with bilinear soil springs or with complete SSI modelling.

8.5 Spatial Variability of Ground Motion

Spatial variability of the ground motion is a well-known phenomenon which may lead to multiple support excitations leading to different ground motion time histories occurring at various supports of the structure. In normal buildings, structures and ordinary bridges, identical support motion is assumed; spatial effects of ground motion are not considered important because of smaller lengths of structures. The equation of motion for multi-support excitation is well known; now available in standard text books on Theory of Vibration or Dynamics of Structures and research papers. The solution of these equations of motion in the time domain leads to the determination of the structural response in earthquakes for this type of motion. The most important aspect is to define time history of multi-support excitation at different support points of the structure using existing methods. The spatial variability of ground motion is important only for long spans. Spatial variability effects involve three main mechanisms: wave passage effect, incoherency effect, and site-response effect. Wave propagation effect represents the time of travel by seismic waves from one support to another. However, when the wave travels in ground the coherency (correlation between the ground motions at the two supports) of waves is lost with travel distance due to multiple reflections and refractions in the propagating medium. The coherency is further lost during propagation through the soil strata below the foundation up to bedrock. Out of these, the site amplification effects, i.e. effect of variation in the soil strata below the different foundations is considered to be most significant **[6].**

Consideration of spatial variation of ground motion is a crucial issue in seismic analysis of long span bridge. **Clause 5.5** (**Table 5.3**) IRC:SP:114 recommends a dynamic analysis with spatial variability of ground motion in case of continuous bridges of length between expansion joints > 150 m. The limit of 150 m length can be interpreted as the distance between the farthest supports (piers/abutments) interconnected by a continuous superstructure. The Eurocode (EC8, Part-2) **[6]** provides two conditions for consideration of variability in ground motion:

(i) The variation of soil conditions at the two supports is such that it can be classified into different site classes (Class A to E defined in **Table 2** of IRC B-3.7 **Annexure-F [5]),**

OR

(ii) The distance between the farthest supports of a continuous deck is greater than *Lg*/1.5, where *Lg* depends on the site class, (which in turn is defined in terms of shear wave velocity) given as:

Site Class	**A, B**	**C**	**D**	**E**
L_g (m)	600	500	400	300

Note: Notwithstanding the above table, as specified in the preface, these guidelines are applicable only for bridges having main span up to 500 m.

It is possible to consider the ground motion variability using time history as well as response spectrum method. In time history analysis (linear or nonlinear) of a cable stayed bridge, the ground motions used in seismic analysis should take into account the effect of wave propagation, loss of coherency with distance, and variation in soil conditions below the different supports. The correlated ground motions at different supports of a cable stayed bridge can be obtained using the procedure described in **Annexure-D** of EC8, Part-2. A simple but approximate approach to consider the wave propagation effect and the ground response effect is by introducing a time lag in the ground motions at bed-rock level and then propagating the bed-rock motions through the soil strata below the corresponding supports to account for the variability is soil conditions. The time lag may be estimated as the travel time for the shear wave for the spacing between the supports and the effective shear wave velocity in the bedrock, whereas 1D nonlinear or equivalent linear wave propagation may be used to consider the effect of soil strata below different supports.

In response spectrum based seismic analysis of a cable stayed bridge, a Multi-Support Response Spectrum (MSRS) method **[7]** can be used to consider the spatial variability of ground motion. The MSRS rule evaluates the mean peak response in terms of the response spectra and mean peak ground displacements at the support points of the structure, and the coherency function that characterizes the spatial variability of the support motions. The MSRS method has been adopted by EC8 (Part-2) **[6].**

The EC8 (Part-2) also provides a simplified Pseudo-Static Method. In this method, the inertia response is accounted for by using a single input seismic action for the entire structure (e.g. a single response spectrum or corresponding time history sets), corresponding to the most severe ground type underneath the different bridge supports. The spatial variation of the seismic action is estimated by pseudo-static effects of appropriate displacement sets, imposed at the foundation of the supports of the bridge deck. The following two sets of horizontal displacements (**8.5.1 and 8.5.2**) are applied separately, in each horizontal direction of the analysis, on the relevant support foundations or on the soil end of the relevant spring representing the soil stiffness. The effects of the two sets need not be combined.

8.5.1 *Set A Displacements*

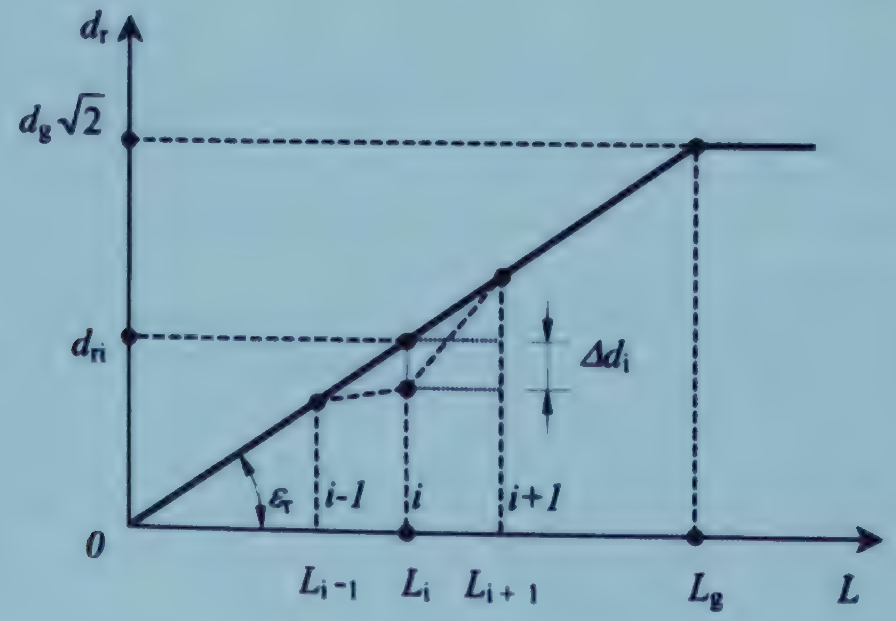


Fig. 8.2 Displacement Set A

Set A consists of relative displacements:

$d_{ri} = \varepsilon_r L_i \leq d_g\sqrt{2}$ where $\varepsilon_r = d_g\sqrt{2}/L_g$

applied simultaneously with the same sign (+ or –) to all supports of the bridge (1 to n) in the horizontal direction considered (**Fig. 8.2**).

where;

d_g is the design ground displacement corresponding to the zone factor ground type of support i, given as 0.57Z, 0.77Z, and 0.95Z (in m), for Soil Types I, II, and III, respectively.

L_i is the distance (projection on the horizontal plane) of support i from a reference support $i = 0$, that may be conveniently selected at one of the end supports;

L_g is the distance beyond which the supports are not interconnected by a continuous superstructure and the ground motions may be considered as completely uncorrelated.

8.5.2 *Set B Displacements*

Set B covers the influence of ground displacements occurring in opposite directions at adjacent piers. This is accounted for by assuming displacements Δd_i of any intermediate support i (>1) relative to its adjacent supports i-1 and i+1 considered un-displaced (**Fig. 8.3**).

$\Delta d_i = \pm\beta_r\varepsilon_r L_{av,i}$

where:

$L_{av,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ of intermediate support i to its adjacent supports i-1 and i+1 respectively. For the end supports (0 and n) $L_{av,0} = L_{01}$ and $L_{av,n} = L_{n-1,n}$;

β_r is a factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports. $\beta_r = 0.5$ when all three supports have the same ground type; $\beta_r = 1.0$ when the ground type at one of the supports is different than at the other two.

ε_r is as defined for set A above. If a change of ground type appears between two supports, the maximum value of ε_r should be used.

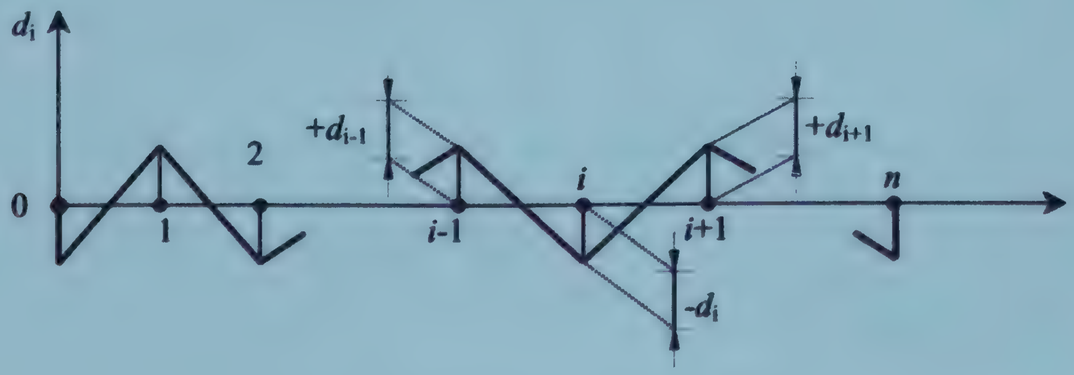


Fig. 8.3 Displacement Set B

Set B consists of the following configuration of imposed absolute displacements with opposed sign at adjacent supports *i* and *i*+1, for *i* = 0 to *n*-1 (as shown in **Fig. 8.3**).

$d_i = \pm \Delta d_i/2; \quad d_{i+1} = \pm \Delta d_{i+1}/2$

In each horizontal direction, the most severe effects resulting from the pseudo-static analyses for support displacements of 8.5.1 and 8.5.2 shall be combined with the relevant effects of the inertia response obtained from response spectrum or time history analysis, by using the SSRS rule (square root of the sum of squares). The result of this combination constitutes the effects of the analysis in the direction considered. For the combination of the effects of the different components of seismic action, the 100:30:30 as per **Clause 4.2.2** of IRC:SP:114 shall be used.

8.6 Seismic Protection Devices

Besides the traditional seismic design for strength and ductility, the incorporation of certain passive and active devices for vibration control in cable stayed bridges can be quite effective in enhancing its ability to resist earthquakes and mitigation of cable vibrations. The seismic protection devices are useful in improvement of seismic behavior and controlling seismic displacements. The Seismic Protection Device basically works on the principle of dissipation of energy; thus, these devices reduce the structural response. Though several types of Seismic Protection Devices are available: Passive, Active, Semi-Active or Hybrid for vibration control; it is necessary to define goal of introducing these devices prior to their use for seismic protection. Some of the goals that can be considered in Cable Stayed Bridges are following: achieve elastic behavior of the bridge for a more frequently occurring earthquake, to improve seismic behavior in a large earthquake, to minimize deck displacements in earthquake, to control vibrations in wind, small earthquakes and traffic loads. The deck displacement without any seismic protection device is usually very high. It is often required to reduce these displacements. In other case the designer will have to accommodate these movements in large expansion joints. The design of seismic protection device and joint design where these devices are introduced should be done together in these bridges. The common passive devices that are recommended for cable stayed bridges are: Viscous damper, Hysteretic damper, Friction damper and Tuned Mass Damper.

The choice of a seismic protection device is guided by several variables. In fact, the design of a device should be an integrated process that should include the force transmitted to the structure, displacements in other directions, devices cost and their installation cost, joints cost and maintenance cost of everything involved during the life cycle of a cable stayed bridge.

One of the key factors in the dynamic behavior of a cable stayed bridge is the connection between the deck and the tower. The current trend in the design of cable stayed bridge is to incorporate seismic devices in the deck tower connection. The objective is to control seismic displacements and the seismic force and to help maintain the towers in elastic range during the earthquake.

CHAPTER - 9

WIND EFFECTS AND AERODYNAMIC DESIGN CONSIDERATIONS

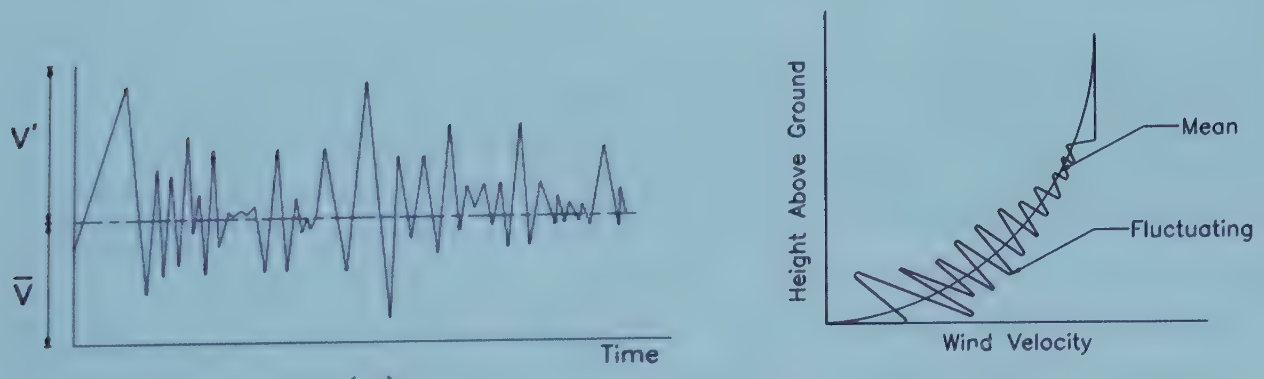
9.1 Introduction

In this Chapter an effort has been made to guide the reader in understanding the basics of wind and its effect on a bridge as it interacts with the wind. Bridges of moderate spans can be designed for this effect by dealing with the problem as a quasi-static one. For longer spans, essentially the aerodynamic approach has to be pursued. Cable-stayed bridges are generally in a span range so as to fall in the latter category. The various kinds of related phenomena that can occur in a cable-stayed bridge are briefly described. Also given are approximate/ empirical relations for assessing the responses of a bridge, besides information and guidelines for its design. It must be emphasized, however, that while based upon the aforesaid information, a preliminary design can be prepared, for the wind-sensitive bridges, the design will require to be proved through further investigation. It will be prudent to include an appropriate period (say, about 6 months) for this purpose in the time-lines of the project.

9.2 Wind Characteristics

Wind flow generation is a result of conducive meteorological conditions, primarily pressure differentials, and manifests itself into a variety of patterns. These are monsoon winds, localized dust storms, thunderstorms, cyclones and tornados, all of which are relevant in the Indian context In the Indian context, except Tornados which are an infrequent phenomenon in India. The east coast experiences greater cyclonic activity compared to the west coast. In cyclones, high-speed winds are accompanied by heavy rains and flooding. Large-scale tropical storms formed over South Pacific and the Indian Ocean are called Cyclones, while they are called Hurricanes when formed over the North Atlantic Ocean and Northeast Pacific, and they are called Typhoons when formed over the Northwest Pacific Ocean.

A trace of wind velocity at any point in space can be generally depicted as shown in **Fig. 9.1** It is obvious that wind is a random time-variant phenomenon, which can be interpreted as the sum of (i) a time-constant, spatially variable mean wind speed, and (ii) superimposed, time and space variable turbulence part. Whereas this is representative of various storm types, the actual structure would vary amongst them. For example, both cyclones and tornados, **Fig. 9.2,** are characterized by swirling winds of very high speed, reaching 300 km/hr. Cyclones are a broadband phenomenon covering hundreds of kilometers, with the major wind velocity content being tangential. Essentially, structures experiencing cyclones will be subjected to similar mean wind profiles like pressure system winds, but the associated turbulence will be higher. The main difference of this storm compared to a pressure system wind is, of course, the large wind speed blowing for several hours but from different directions caused by the rotational component of the storm. These emanate from the sea, and, cover vast areas of land near the coasts. Tornadoes, on the other hand, are a narrow band phenomenon, covering no more than a few hundred kilometers transversely. These are essentially an in-land activity, covering comparatively small areas. While a tornado generates very high tangential wind velocity, it is also accompanied by substantially potent updraft and radial velocities. Considering the rare occurrence of tornadoes in India, though the associated speeds are high, these speeds are associated with a very high return period. The design cannot be economically viable considering such rare occurrences, and hence shall not be considered in the design for bridges. Hence, for the design, consider the speeds in the IRC:6 **[1]** which is in line with IS:875 (Part 3) **[2]** covering more frequent monsoons and cyclones.



(a)

(b)

Fig 9.1 Depiction of Wind Velocity in Time – Space Dimension

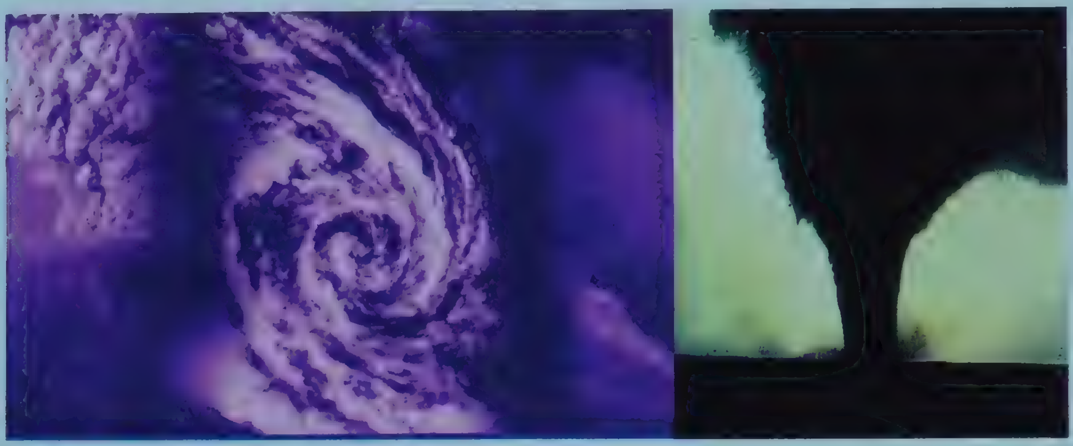


Fig. 9.2 Cyclones and Tornadoes

The wind being an inherently dynamic phenomenon, all structures will undergo a dynamic response to a smaller or greater degree, when these obstruct its flow. Structures that are comparatively stiff (say, with natural frequencies greater than 1 Hz), will respond in a quasi-static manner, and can be dealt with accordingly. The quasi-static approach given in IRC:6 **[1]**, apparently addresses shorter span bridges. Longer spans and more flexible ones, such as cable-stayed bridges, which will fall into the wind–sensitive category, are expected to experience a predominantly aerodynamic response.

9.3 Dynamic Characteristics

The dynamic characteristics of cable-stayed bridges for wind-induced oscillations invoke great interest. These, being flexible structures, have low natural frequencies, generally less than 1Hz, and, are consequently more prone to vibrations. The vibration of cable-stayed bridges may involve a single member (deck, cable or tower) or the entire structure. Cable-stayed bridges tend to have very closely spaced natural frequencies and often significant structural coupling of flexural and torsional modes of vibration due to separation of mass center and stiffness center of the deck.

The frequency of vertical oscillation of bridge deck depends on the stiffness of stay cables in the vertical direction, stiffness of the girders, and mass of the structure. The stiffness mainly depends on main span length, the ratio of side span to main span length, the ratio of pylon height above bridge deck level to main span length, and the cross-sectional area of stay cables. The torsional frequency of the bridge deck depends on its torsional stiffness, e.g. a box girder is stiff while an "open" cross-section is weaker in torsion. Towers can also affect the torsional behavior of the deck. However, the tower is effective in torsion only if the tower shafts are tied with cross beams. A-shaped pylons with inclined cables provide higher resistance to torsional oscillations of the deck than those with an H-shape, with double plane cable arrangement (**Fig. 3.2** of these guidelines), though the latter have been frequently deployed in bridges of up to 500 m span.

Cables arranged in a single plane and connected to the deck along the centerline provide no resistance to the torsional rigidity of the deck. Such bridge decks have low torsional natural frequencies and low torsional damping. On the other hand, cables arranged in two planes and connected to the deck along its two edges provide a high degree of resistance to torsional oscillations. The resistance to lateral bending mode is mainly offered by the deck. The longitudinal oscillation of the deck is governed by the resistance offered mainly by the tower where bearings in the vertical plane can be provided to restrain transverse movement without impeding longitudinal movements.

Individual cables being flexible, possess very low structural damping, with a very much lower mass in comparison to deck or pylon. These often get excited in the vertical plane.

The vibration characteristics of a cable-stayed bridge, after completion, are quite different from that of the bridge during construction. The various procedures for erection, which include quite commonly the cantilever mode, are discussed in **Chapter 5, Section 5.2** of these guidelines.

During the various erection stages, mass (including the construction equipment) and stiffness are changing drastically, which affects the vibration characteristics and hence the corresponding wind responses. Therefore, often the bridge may be more vulnerable during the construction stages, in comparison to the completed bridge.

9.4 Wind Effects

9.4.1 *Dynamic Wind Effects on Bridge Deck*

The major wind induced problem in cable bridges is on account of the oscillations in the deck. These are classified broadly as limited amplitude oscillations and divergent amplitude oscillations **Fig. 9.3**. Prolonged limited amplitude oscillations could cause unacceptable stresses and fatigue damage in bridge components and discomfort to the users, whereas divergent amplitude oscillations may lead to distress or even failure.

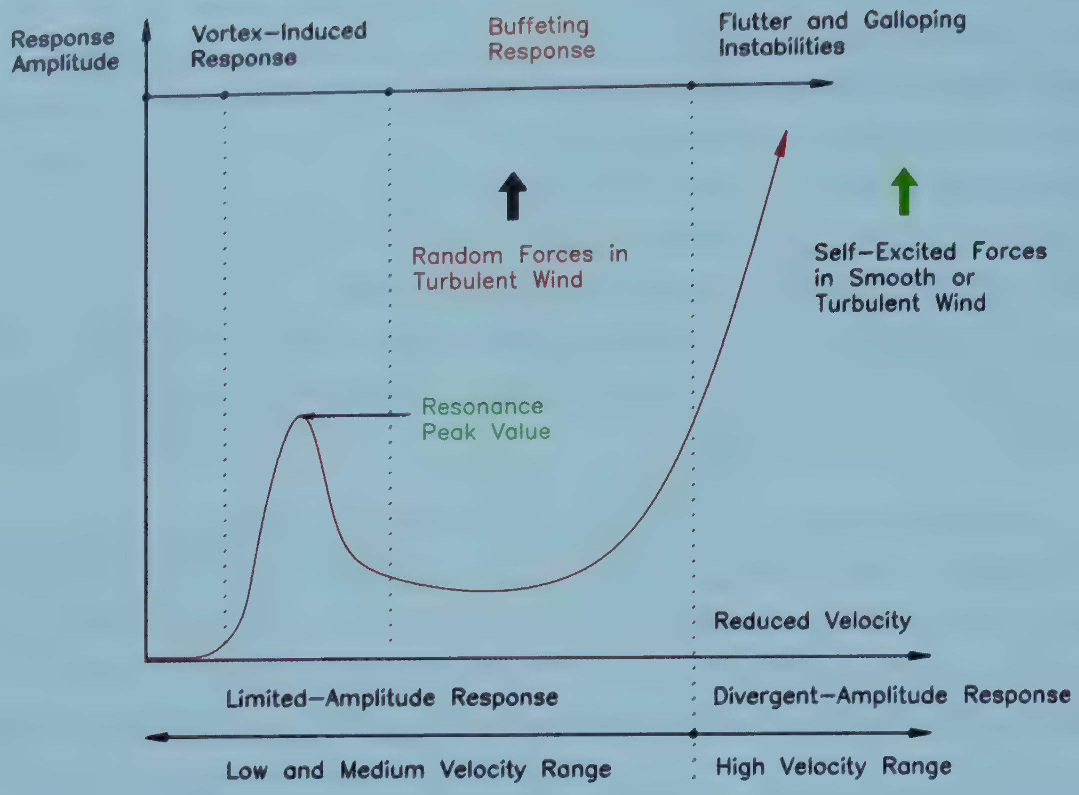


Fig. 9.3 Various Forms of Aerodynamic Response Exhibited by the Deck in Different Wind Velocity Ranges (Wind Velocity Range indicates Reduced velocity, Vre = V/fB, where f is frequency of vibration and B is a typical dimension)

9.4.1.1 Limited Amplitude Oscillations

These can be caused either by vortex induced oscillation or buffeting.

Vortex Induced Oscillation: These oscillations are caused due to the shedding of vortices alternatively from the top and bottom surfaces of the deck. The vortices produce periodic cross wind forces which cause oscillations either in vertical or torsional modes. The frequency of shedding of complementary pairs of vortices is expressed in non-dimensional form by the Strouhal number 'S', (S= fD/V) which is typical for any particular geometrical shape, where *f* is the vortex shedding frequency, *D* is the reference dimension of cross-section and *V* is the wind speed. When the frequency of vortex shedding, f, coincides with a natural frequency of the bridge deck, large amplitude oscillations may occur. This type of oscillation may occur over one or more narrow ranges of wind speeds.

Buffeting: The wind in the vicinity of earth's surface is turbulent. Turbulence causes buffeting and leads to fluctuating forces and moments on the bridge components. This action of the turbulent wind is experienced over a wide range of frequencies. The bridge structure must be analyzed for such buffeting responses.

9.4.1.2 Divergent Amplitude Oscillations

In divergent amplitude oscillations, the amplitude can continue growing until possible collapse. This is a consequence of loss of damping and/or stiffness arising from interactions of aerodynamic forces with changing deformations of the deck. These oscillations mainly manifest in the form of galloping and flutter due to instabilities in the vertical, lateral and torsional motions either in isolation or in combination. Single degree of flutter is also called galloping.

The phenomenon of flutter arises in the form of single degree of freedom motion or coupled motion. Classical flutter arises due to the coupling of torsional and vertical motions. Flutter may ultimately lead to collapse unless it is restricted by the nonlinearity of the structure. Collapse of the Tacoma Narrows suspension bridge in the year 1940 was a landmark failure on account of flutter. The critical speed for flutter of a deck section can be arrived at by model testing in the wind tunnel. Alternatively, by complex eigenvalue analysis of the bridge the theoretical critical wind speed for flutter can be estimated. Theoretical analysis requires motional derivatives (flutter derivatives) for the deck cross-section which are again obtained from wind tunnel testing.

The cause and effects of wind induced vibrations are listed in **Table 9.1** [after ref. 3]

Table 9.1. Wind–Induced Vibrations

Types of Vibration	**Turbulence Induced (buffeting)**	**Vortex Induced**	**Motion Induced (Flutter)**
Cause	Random fluctuations of wind speeds in atmospheric boundary layer	Regular shedding of vortices from the body	Rhythmic motion of body in the flow
Excitation Forces	Along and transverse to wind direction	Mainly transverse to the wind direction	Mainly transverse and torsional
Effect	Dynamically amplified internal forces	Sharp resonance	Self - excitation instability

9.4.2 *Dynamic Wind Effects on Tower*

The dynamic wind effects on tower could be in the nature of along wind oscillations and on account of vortex induced oscillation (limited amplitude). Galloping is possible but is not expected to be commonplace. It has been found that the rectangular sections with width to depth ratio less than five are prone to galloping. Further, it may be important to consider galloping in the design of towers during erection before the cables and decks are installed.

9.4.3 *Stay Cable vibration under Wind*

Stay cable strands with a circular cross section, can oscillate at certain velocity of wind. This is called the ‘singing’ of the cable. Likewise, when provided as a group of 2 or 4 ropes or strands in place of a single rope or strand, in order to use smaller sizes, there is the possibility of wake vibration in the leeward element, **Fig. 9.4**

Another form of wind induced vibration that can occur in the stays when rivulets flow down, **Fig. 9.5** the stay due to the falling of rain and there is simultaneously a flow of wind. This is called rain-wind vibration, and, is a greater possibility in longer cables.

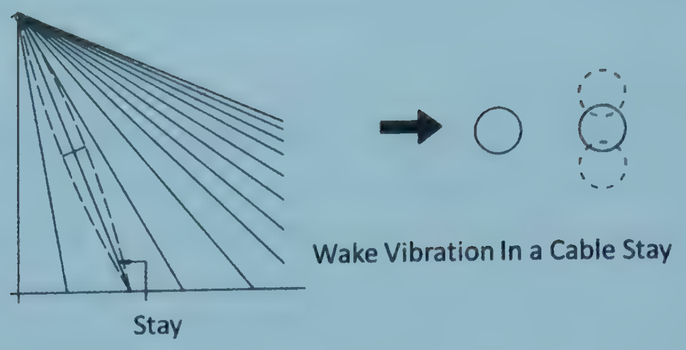

Wake Vibration In a Cable Stay
Stay

Fig. 9.4 Wake Induced Vibration of a Stay Cable

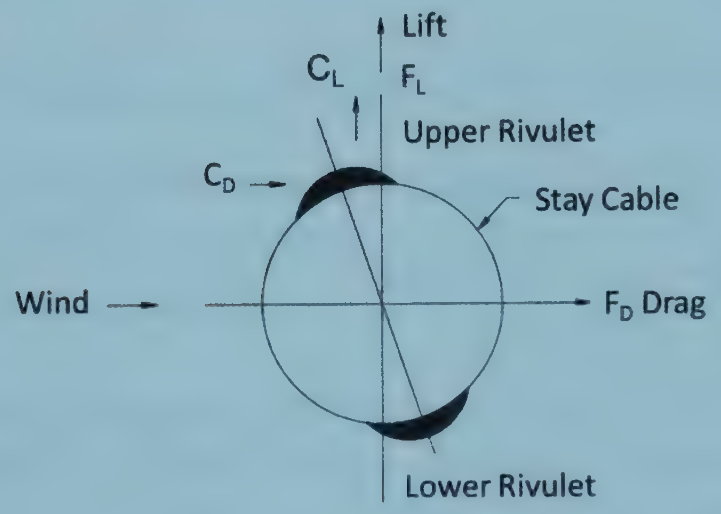

Lift
C_L
F_L
Upper Rivulet
C_D
Stay Cable
Wind
F_D Drag
Lower Rivulet

Fig. 9.5 Rain-Wind Vibration- Formation of Rivulet

Therefore, wind actions that shall be considered for the design of cable stayed bridges, besides the direct or quasi-static wind load, are as follows:

i) Flutter

ii) Galloping

iii) Vortex induced oscillation

iv) Buffeting response

The wind actions may produce the following effects:

i) Excessive forces, stresses or deformation of the bridge or its elements

ii) Repeated dynamic stress or deformation causing fatigue of bridge or its elements

iii) Aerodynamic instability

iv) Excessive dynamic movement causing discomfort to the users.

9.5 Principles for Wind Resistant Design

The wind resistant design principles for cable stayed bridges consist of the following:

1) With design wind loading and other actions, the bridge shall have sufficient strength, rigidity and shall meet the requirements of static stability.

2) During the service life of a bridge, it shall not be prone to divergent amplitude oscillations such as flutter and galloping

3) A low amplitude of oscillations due to vortices/ buffeting shall meet the requirements of traffic safety, structural fatigue and comfort of road users.

4) Wind resistant design shall be achieved by considering geometrical, structural or mechanical countermeasures.

5) The wind tunnel test is one important method to carry out the wind-resistant design of bridge structures (**Section 9.10.2**).

9.6 Conceptual Design Practices

The following aspects may be considered while selecting the deck, tower and cable type and geometry so that the wind effects are minimized. **[4]**

i) For long-span cable-stayed bridges, deck, tower and cable shall be chosen with consideration for wind-resistant stability.

ii) Cable-stayed bridges provided with single plane cable system along the centre line have usually a low torsional resistance, and are likely to be prone to aerodynamic instability named torsional divergence. For highway bridges with single plane cable system, single box girder is a standard solution for deck on account of its high torsional rigidity.

iii) For highway bridges with double plane system, depending on the span and width of bridge the selection of deck can be made (**Chapter 4**).

iv) For spans up to 200 m with deck width in the range of 15 m to 20 m, simple solid concrete slab without edge beams can be a good solution, while for wider decks between 20 m to 25 m, and spans in the range of 200 m to 500 m, T or I-beam supported deck with cross beams is expected to be more suitable.

v) Concrete cable stayed bridges with cables in two-planes along the edges are considered to be stable with respect to wind action if $B \geq 10d$ (where B= breadth of deck and d= deck depth). Alternatively, if $B < 10d$, then bridge deck with wind nose or fairing is preferred. Concrete decks with $B \geq L/30$ are considered favourable under wind action.**[5]**

vi) Steel bridges with main spans more than 400 m, provided with lightweight deck are more susceptible to wind-induced oscillations and A-shaped tower is preferred. Steel decks with width B≥ L/25 is favourable with respect to wind action, alternatively provide wind nose or fairing if B< L/25.

vii) In the case of a truss-stiffened deck, plate girder supported decks, box –girder decks, good aerodynamic performance of bridge deck could be achieved by the use of shallow sections, closed sections, edge streamlining, and other minor and more subtle changes to the cross-sectional geometry.

viii) To increase the structural safety of bridge deck due to vortex induced fatigue, it is advantageous to make the onset of wind velocity for vortex excitation as high as possible. To achieve this condition, the bridge deck section should be as flat as possible but it should have sufficient stiffness to increase the natural frequency.

ix) Buffeting response of bridge deck to fluctuating wind depends on the steady-state drag (C_D), lift (C_L) and moment (C_M) coefficients as well as the derivative of lift ($dC_L/d\alpha_w$) and moment coefficient ($dC_M/d\alpha_w$) corresponding to angle of attack of wind. By selecting a cross-section for the bridge deck such that these aerodynamic design parameters and their derivatives are minimized, it could lead to a lower buffeting response.

x) Flutter stability of cable stayed bridge deck could be greatly increased by separation of vertical and torsional modes (f_t/f_v >2, where f_t and f_v are the first torsional and vertical mode frequencies of bridge deck respectively), high torsional rigidity for deck, and by adopting stream-lined bridge deck cross-sections.

xi) Provision of high torsional stiffness by the use of box sections for bridge deck and
A-shaped towers can help in raising the critical wind speeds for flutter to safe levels. The A- shaped tower gives a triangular shape of the cable plane and the deck, which increases torsional rigidity.

xii) For long-span bridges located in zones of high winds, A- shaped tower is the ideal solution to achieve wind stability. The upper level and deck level struts connecting the tower legs further help to improve the resistance in torsion.

xiii) Cables used in conjunction with cable pipes with surface treatment are preferred.

9.7 Desktop Study for Checking the Aerodynamic Susceptibility (Stage1)

The wind effects on cable stayed bridges (depending on span arrangement) consist of static and dynamic wind effects. The methodology to be adopted to assess the susceptibility of various structural components of CSB is given below.

Step 1: Structural and Wind Characteristics

The input required for the desktop study are (i) Main span length of the bridge (L) (ii) Deck width (B) and (iii) Deck depth (d) at mid span and at the location of pylon.

For assessing the wind characteristics at the location of the bridge the following parameters are considered. (i) Level of bridge deck above the mean ground level (ii) Basic wind speed zone depending on the location of bridge (iii) Terrain category at the location of bridge as per IS:875: Part 3:2015 **[2]** (iv) Turbulent intensity (I_u) at the level of bridge deck, which depends on the terrain category can be computed. Also, compute the design wind speed at the location of bridge, taking into account the effects of terrain, topography and cyclonic conditions (V_d based on 3 sec gust) at the bridge deck level as per IS:875 (Part 3) :2015 **[2].**

Step 2: Checking the susceptibility for dynamic wind action as given in Table 9.2 for bridge deck

Table 9.2 Checking the Susceptibility of Bridge Deck to dynamic wind effects (based on ref. [6])

Type of Bridge deck cross-section	Type of Wind induced oscillations of deck	Susceptibility Parameter	Limiting value	Inference	Further Action Required
Closed solid deck cross-section	Single degree of flutter in Torsion	L Vd/B	L Vd/B > 780	Deck is susceptible to torsional flutter	See note (i) to (iii) below. Also, perform check as stipulated in **Section 9.8** if main span L > 300 m
Truss, open solid deck cross-section			L Vd/B > 530		
Truss, open solid, closed solid deck cross-sections	Galloping	LVd/B and B/d	For composite decks LVd/B > 475 and B/d< 5	Deck is susceptible to galloping	See note (i) and (iv). Also, perform check as stipulated in **Section 9.8**
Truss ,open solid, closed solid deck cross-sections	Vortex induced oscillation	LVd/B and Iu	LVd/B > 275 and Iu < 0.2	Deck is susceptible to vortex induced oscillation	See Note (i) and ((vi)and perform check as stipulated in **Section 9.8**
For checking the susceptibility of bridge deck against buffeting, see note (i) and (v) below. When the fundamental frequencies in both bending and torsion estimated for the bridge components are greater than 1.0 Hz, the effects of turbulence response shall be ignored.					

Notes

i) If the bridge deck is susceptible to wind action as shown in **Table 9.1**, a free vibration analysis of the bridge subjected to dead load and SIDL is carried out using 3D-modelling with appropriate boundary conditions.

ii) Identify the respective vibration modes of deck, tower and cables (by examining the mode shapes of vibration) with frequency values less than 1.0 Hz, which are susceptible to wind induced oscillations. Check the ratio of first torsional to vertical mode of bridge deck and if this ratio is more than 2, the possibility of occurrence of classical flutter is ruled out. Otherwise, the wind speed for initiation of classical flutter shall be estimated and compared with design wind speed at bridge site.

iii) The critical wind speed for classical flutter V_f shall be computed from the reduced critical wind speed (V_{Rf}) **[7]**

$V_{Rf}= V_f/(f_t B)$ (.....Eq.9.1)

where $V_{Rf} =1.8 (1-1.11(f_v/f_t)^2)^{1/2} (mr/\rho B^3)^{1/2}$ (.....Eq.9.2)

r- polar radius of gyration of effective bridge deck cross-section at the centre of main-span (polar second moment of mass/mass)$^{0.5}$

The bridge deck shall be stable with respect to divergent response in wind storms if Vf >1.65 design wind speed (V_d) (based on 3 sec gust) at bridge location.

Compute the critical wind speed for initiation of single degree of freedom (sdof) flutter (V_{crf}). When the width-to-depth ratio of the bridge deck section B/d=4~8, the flutter critical wind speed of bridge with main span less than 300 m may be calculated according to the following equation:

$V_{crf} =5f_t \cdot B$, where f_t is the first torsional frequency of bridge deck in Hz.

If the estimated critical wind speed for flutter is greater than 1.65 times design wind speed (V_d) at bridge location (3 sec), then occurrence of torsional flutter could be ruled out.

iv) Estimate static coefficients for bridge deck from CFD studies or wind tunnel tests on sectional model at different angles of attack. The susceptibility of bridge deck to galloping shall be evaluated by checking signs of the time-averaged static wind coefficients for lift and moment at zero angle of attack. A negative slope of static lift (or moment) coefficient usually indicates a tendency for galloping. Also it can be checked using Den Hertog Criterion i.e., at wind angle $\alpha =0$, if $(C_d +(dC_L/d\alpha) <0$, where: α is the angle of attack in degrees, C_L is the aerodynamic lift coefficient and C_D is the aerodynamic drag coefficient of bridge deck at a wind angle zero.

v) The wind in the vicinity of earth's surface is turbulent. Turbulence causes buffeting and leads to fluctuating forces and moments on the bridge components. This action of the turbulent wind is experienced over a wide range of frequencies. For bridge decks with fundamental frequencies in respective bending (vertical), torsion or lateral directions <1.0 Hz buffeting response to be considered. Alternatively for bridge decks with span-to-depth or span-to-width ratios greater than 30, buffeting effect must be taken into consideration for aerodynamic design.

vi) Identify the vertical modes of bridge deck with frequency less than 1.0 Hz by inspecting the various mode shapes of vibration, and, compute the respective critical wind speed for vortex excitation for vertical modes of bridge deck with frequency less than 1.0 Hz, $V_{cr}= f_v\ d/S$, where f_v is the vertical frequency of bridge deck, 'S' Strouhal Number for box -girder bridge deck, which is in the range 0.1 to 0.15, for truss /open solid deck the Strouhal Number is in the range 0.1 to 0.2.

If the computed V_{cr} values are lower than design wind speed at the deck level, then these modes are prone to vortex excitation in the vertical direction. Once vortex-induced oscillations have been established at a given wind speed, the vortex shedding frequency may "lock" into the bridge natural frequency as long as the wind speed does not vary significantly. Therefore, vortex shedding often persists over a narrow range of wind speeds. It may be noted that vortex shedding is most noticeable in smooth flows and tends to decrease as the turbulence of the wind increases.

In addition, follow the guidelines given below:

a) The effect of vortex resonance may not be considered when the fundamental frequency of structural component is larger than 5 Hz.

b) Twin box-girder bridges with an open gap are particularly sensitive to vortex induced motions. Vortices shed from the windward box combine with the vortices shed from the leeward box to increase the wind load on the leeward box, making a twin box configuration more sensitive to vortex induced motions than a single box configuration.

The Strouhal Number $S = f\ d/V_{cr}$ defines the wind speed when vortex excited motions may occur. See **Table 9.3** for Strouhal number of a twin box-girder.

The gap ratio is defined as the gap width divided by the overall width of the twin boxes.

Table 9.3 : Gap Ratio Vs Strouhal Number

Case	Gap Ratio	Strouhal Number
1	0.000	(no vortex induced motion)
2	0.025	0.13
3	0.160	0.23
4	0.27	0.27
5	0.35	0.28

c) The vortex induced oscillatory amplitude in vertical direction of a box girder bridge deck may be computed according to the following equation [8]:

$$h_a = \frac{E_h E_{th}\, B}{2\pi m_r \zeta_s} \qquad \text{(.....Eq.9.3)}$$

where, $m_r = \frac{m}{\rho B^2}$, $E_h = 0.065\, \beta_{ds}\, \frac{d}{B}$, $E_{th} = 1 - 1.5\, \beta_t \sqrt{\frac{B}{d}}\, I_u^2 \geq 0$ and $I_u = \frac{1}{ln\frac{Z}{z_0}}$

h_a—The vertical vortex induced oscillatory amplitude (m);

m—The linear mass of bridge (kg/m). It may be taken as average value at 1/4 span as for the bridge with variable cross-section along with half of the mass of stay cable system

ζ_s— critical damping ratio of bridge deck

β_{ds}—The shape correction coefficient, which shall be taken as 1 for a bluff -body with width larger than ¼ of effective height.

B—The width of deck (m)

d—The depth of deck (m)

β_t—The coefficient shall be taken as 1

I_u—The turbulence intensity;

Z—The reference altitude of deck (m)

z_0—The terrain roughness height at bridge site (m) for different terrain categories (TC1 to TC 4) which may be selected according to IS:875: Part 3:2015 **[2]** till it is included in IRC:6 **[1]**

Check whether amplitude of oscillation due to vortex excitation (h_a) of bridge deck is less than $0.04/f_v$ otherwise it may cause discomfort to users. If yes, then, consider geometric modification of bridge deck or by installing a TMD

d) Check the static stability

The wind velocity, called the divergence velocity V_{dg}, corresponds to the occurrence of a static stability problem. The divergence wind speed can be calculated by using the aerodynamic moment coefficient C_M of the deck for varying wind angles of attack α, by using Eq. 9.4 **[9]**.

$$V_{dg} = \sqrt{\frac{2\,K_\alpha}{\rho B^2 \frac{dC_M}{d\alpha}}} \qquad \text{(.....Eq.9.4)}$$

For bridge deck to be stable under wind action, $V_{dg} > 2\,V_d$

Step 3: Susceptibility of Tower to Dynamic Wind Action

Towers of cable stayed bridges may be prone to dynamic wind action, i.e., vortex induced oscillation or buffeting, if the frequency of lateral or longitudinal mode of towers from note (i) below **Table 9.1** is less than 1.0 Hz or When ratio of height to minimum lateral dimension of tower is more than 5.0. The transverse or lateral and longitudinal modes of vibration of pylon are identified by examining the mode shapes of vibration obtained from the free vibration analysis.

For estimation of vortex induced oscillation of upper pylon, Strouhal number (S) of 0.12 may be used

$V_{cr}=f_p D_p/ S$ (.....Eq.9.5)

If the height of free-standing pylon (i.e., during construction stage) is more than 30 m, then safety against dynamic wind shall be considered.

Step 4: Check for Wind induced cable vibration

The wind-induced vibration of stay cables can be categorized as vortex induced oscillation, rain-wind induced vibration and wake galloping. The cable vibrations are likely to cause the fatigue due to secondary bending at the region near cable to deck /tower anchorage, but also cause discomfort to bridge users.

Therefore, it is necessary to check the susceptibility of vibration of stay cable and incorporate countermeasures to mitigate the dynamic wind effects.

Dynamic effects of the wind especially in cables with length greater than 65 m shall be considered as follows.

a) Vortex-induced Vibration (VIV)

VIV is due to the alternate shedding of vortices from opposite sides of the strand, inducing a periodic force in the cable. As steady wind is required for this effect to occur, the most damaging vibrations generally occur at low wind velocities. Vortex shedding is expected to occur when its frequency becomes approximately the same as the natural frequency of a cable, that is to say, a wind velocity approaching the resonance inducing velocity. In order to avoid this phenomenon, the natural frequency of cables should not be the same as the vortex-shedding frequency described below:

$V_{cr}= f_c d_c/S$ (.....Eq.9.6)

Where

f_c- frequency of vibration of the cable obtained from free vibration analysis

V_{cr} – Critical wind speed for vortex excitation of extradosed cable (m/s)

S - Strouhal number 0.2 for circular cable

d_c - outer diameter of the cable (m)

Also, the Scruton number (Sc) for the extradosed cable shall be computed using Eq. 9.7. The Scruton number for stay cables without any dampers is typically Sc less than 1. However, if S_c is more than 2.5, then vortex induced oscillation is restricted.

VIV can be mitigated by reducing or eliminating the underlying excitation along the lengths of the cables that drives cable oscillation by introducing projections, or texturing, of the cable pipe. This roughens the surface of the cable and presents an irregular surface to the wind flow.

b) Rain-wind induced vibration

Natural frequency of cables shall be obtained from three-dimensional free vibration analysis of the system. Rain-wind induced vibration occurs in cables with smooth surface protective tube and with natural frequency in the range of 1 to 3 Hz . The wind velocity range is around 6 to 18 m/s. The wind must be from an oblique direction, between 30° and 80° from the perpendicular to the cable, so that the wind will tend to lift the cable. Below the stated wind speed range, the top rivulet does not form because the wind is insufficient to prevent it from running down the side of the cable. Above the range, the wind forces tend to blow the rivulets off the cable. Wind turbulence also prevents the rivulets from forming.

In order to avoid the onset of rain–wind oscillation it has been suggested that the Scruton number (S_C) for the cable should be at least 10 according to the following formula specified in **[7]**

$$S_C = \frac{m\xi}{\rho {d_c}^2} \geq 10 \qquad (\ldots\ldots Eq.9.7)$$

where

m - cable mass per unit length (kg/m) $\xi = \frac{\delta}{2\pi}$ and δ is logarithmic decrement of cable. For preliminary estimation, the critical damping ratio may be taken as 0.1% for all type of cables. Alternatively, the critical damping ratio values certified by the cable manufacturer shall be used.

ρ is the air density (kg/m^3)

d_c is the cable diameter (m)

Rain–wind induced oscillation is therefore unlikely to occur if the Scruton number satisfies Eq.9.7.

Where cable pipes have effective surface texturing (special helix or dimpled stay pipes) the above limit may be relaxed to 5, i.e. $S_C \geq 5$,

c) Wake galloping

When two cables are aligned parallel and close to each other, the downstream cable can be forced into unstable oscillations which are called wake galloping. Occurrence of the wake galloping depends on spacing between the cables. Wake galloping is reported to be observed when spacing between the cables is about 2 to 5 times of cable diameter, d_c. It is found that the condition does not occur for cable spacing larger than $5d_c$

9.8 Desktop study for Predicting Aerodynamic Susceptibility (Stage 2)

Wind resistant design for short to medium span bridges may be carried out by considering quasi-static wind effects, whereas longer span bridges necessarily exhibit a marked aerodynamic behavior. Most cable-stayed bridges, being inherently flexible, are in the latter category. Since geometrical proportions of the bridge and the wind climate at its location too will influence the behavior of the bridge, a strict dividing line based on span alone may not be appropriate. Perhaps a dividing span length between 150 and 250 m may be more prudent from the point of view of these guidelines. Even for wind-sensitive longer spans, the quasi-static approach may be used for preliminary design and proportioning, before aerodynamic considerations are addressed.

Long span bridges; especially cable suspended bridges being flexible are prone to wind induced oscillations, especially due to gusts. These bridges undergo several forms of aerodynamic oscillations, which may result in motions in isolated vertical bending or torsional modes, or sometimes in coupled vertical bending and torsional modes.

Depending on the nature of excitation, the motion may of the following types for the bridge deck:

i) Limited amplitudes due to vortex-induced oscillations or turbulence in approaching wind, termed as buffeting, cause unacceptable stresses or fatigue damage. These type of oscillations shall be considered, during the design.

ii) Divergent amplitudes increasing exponentially to larger values, due to flutter or galloping, which must be avoided.

iii) Non-oscillatory divergence due to a form of aerodynamic torsional instability which must be avoided.

Aerodynamic susceptibility of the bridge may be checked as follows:

Method 1: Check whether the bridge is aerodynamically sensitive by computing the aerodynamic susceptibility parameter (P_b) **[7]**. It is stipulated that,

i) If the parameter P_b, given in Eqn. 9.8, is between 0.04 and 1, the bridge is prone to aerodynamic oscillations.

$$P_b = \left(\rho \frac{B^2}{m}\right)\left(16 \frac{(V_H)^2}{BLf_v^2}\right) \qquad \text{(.....Eq.9.8)}$$

where

ρ - mass density of air (1.2 Kg/m^3)

B - deck width edge to edge

L - main span length of the bridge

V_H - hourly mean wind velocity corresponding to 50-year return period

f_v - natural frequency of the bridge in vertical bending mode in Hz

M - mass per unit length of deck

Case1: If $P_b < 0.04$ the bridge will have insignificant effect of all forms of aerodynamic excitation

Case 2: If $0.04 \le P_b < 1.0$ then the bridge deck is aerodynamically susceptible

Case 3: If $P_b > 1.0$ the bridge deck is potentially very susceptible for aerodynamic excitation (bridges with main span more than 250 m are likely to fall in this category)

Method 2: Bridges and bridge components are deemed wind-sensitive **[10]**, when the mass damping parameter $S_w < 1.0$,

where $S_w = (Sc)\,(B/L)\,(B/d)\,(f_v B/\,k_1 \times V_b)$ (.....Eq.9.9)

and $Sc = m\zeta/\rho d^2$

m - mass per deck length

ζ - structural damping ratio

ρ - air density

d - deck depth

B - deck width edge to edge

L - main span length

f_v - lowest frequency in vertical mode

V_b – basic wind speed at bridge location

It is imperative that even if one of the above methods indicates a need for aerodynamic investigations, these should be pursued. Furthermore, whatever may be the indication, for bridges with more than 250 m main span, aerodynamic investigations including wind tunnel testing should be carried out.

If a bridge site is windy, such as locations on the coasts of India, or the hilly regions where deep valleys can experience accentuation of wind speed, the project should, in any case, consider including aerodynamic investigations. It will be prudent to carry out a wind climate assessment which will provide more detailed information on the wind climate around the bridge site. This will be expected to follow a 'desktop analysis' for the bridge, and, if so indicated a wind tunnel study be undertaken.

9.9 Data Required for Aerodynamic Design

9.9.1 *Geometric Details*

Geometric details of bridge deck, cable and tower are required for modelling purpose. Also, it should include the geometric details of railing, crash barrier, noise barrier and other components which are essential for numerical modeling /wind tunnel studies.

9.9.2 *Material Properties*

The material used for construction of deck, tower and cable, their density ,modulus of elasticity, mass per unit length, poisson's ratio etc. are needed.

9.9.3 *Damping Ratio for Bridge Deck and Tower*

The following structural damping ratios (percentage of critical damping) depending on the material to be used, are recommended for analysis of wind effects, including aerodynamic stability analyses:

- Steel bridges – 0.5%
- Composite steel/concrete bridges – 0.7%
- Concrete bridges – 0.9%

For bridges under construction, reduce the above recommended values by 0.2%

9.9.3.1 Damping Ratio for Cables

For preliminary estimation, the critical damping ratio may be taken as 0.1% for all type of cables. For better estimates see **Table 9.4** or the values certified by the cable manufacturer.

Table 9.4 Typical damping of Stay Cable System

Cable Stay Type	Critical damping ratio
Grouted stay	0.08 to 0.13%
Locked coil	0.15 to 0.3%
Stay cable provided with HDPE duct, galvanized strands and filled with wax	0.1 to 0.18%
Stay cable provided with HDPE Duct and individually protected strands (Freyssinet)	0.1to 0.20*

*Damping factor = -6×10^{-4} L + 0.24 (where L is length of stay in m)

9.9.4 *Wind Characteristics at Bridge Site*

To determine wind effects on cable-stayed bridges, it is stipulated that the basic relevant information about wind will be obtained. Bridges in the cyclonic regions will be treated as life-line structures of post-disaster importance. Since wind clause of IRC:6 **[1]** is based on IS:875: Part 3 **[2]**, the same shall be referred for description of wind.

The wind speeds as found in contemporary literature or documents are commonly based on different averaging times and return periods. In this context, certain conversion factors (k_1 Factors) have been included for the design. For the structural design of cable stayed bridge a return period of 100 years and for the aerodynamic stability check the return period of 10,000 years shall be used. For construction stage analysis, a return period of 10 years shall be used. **(Table 9.5)**.

Table 9.5. Multiplier Factors, k_1, for Velocity V_b for Different Return Periods (V_b m/s for Return Period of 50 years as given in [2])

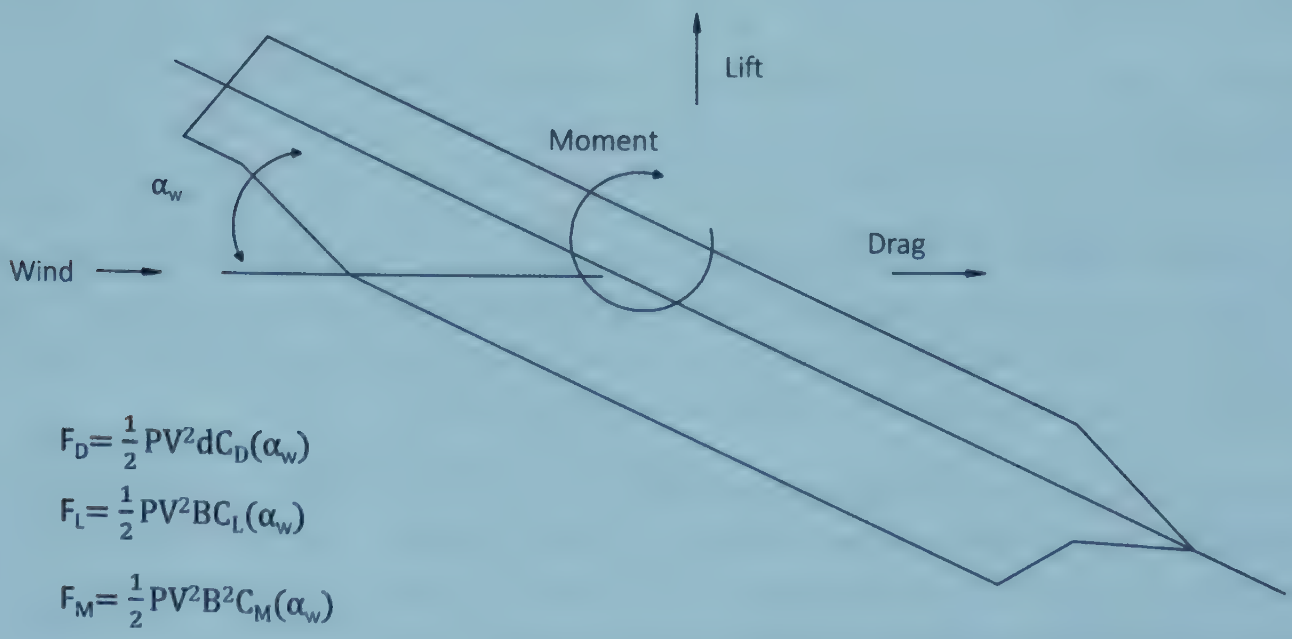
Return period of wind (years)	k_1 factor for basic wind speed (m/s)					
	33	39	44	47	50	55
5	0.82	0.76	0.73	0.71	0.70	0.67
10	0.87	0.83	0.81	0.8	0.79	0.77
25	0.94	0.92	0.91	0.91	0.9	0.89
50	1.0	1.0	1.0	1.0	1.0	1.0
100	1.05	1.06	1.07	1.07	1.08	1.08
500	1.17	1.22	1.25	1.27	1.28	1.30
1000	1.23	1.29	1.33	1.35	1.37	1.40
10000	1.40	1.52	1.59	1.63	1.66	1.71

9.9.5 *Design Wind Load*

Wind load on respective components shall be assumed to be distributed on the area exposed to the wind. The wind load on a bridge shall be calculated for (i) Bridge deck between expansion joints (with and without live load at service and construction stages) (ii) Tower (iii) Stay cable system

9.9.5.1 Bridge Deck

Static wind forces on the deck F_D, F_L and F_M can be computed in terms of the force coefficients C_D, C_L and C_M **Fig. 9.6** representing drag, lift and pitching moment respectively for the deck. These coefficients are to a great extent dependent upon the shape of the deck as well as the angle of incidence of wind (measured in the vertical plane). Some typical deck cross-sections and the corresponding values of C_D are shown in **Fig. 9.7.**



$$F_D = \frac{1}{2} P V^2 d C_D(\alpha_w)$$

$$F_L = \frac{1}{2} P V^2 B C_L(\alpha_w)$$

$$F_M = \frac{1}{2} P V^2 B^2 C_M(\alpha_w)$$

Fig. 9.6

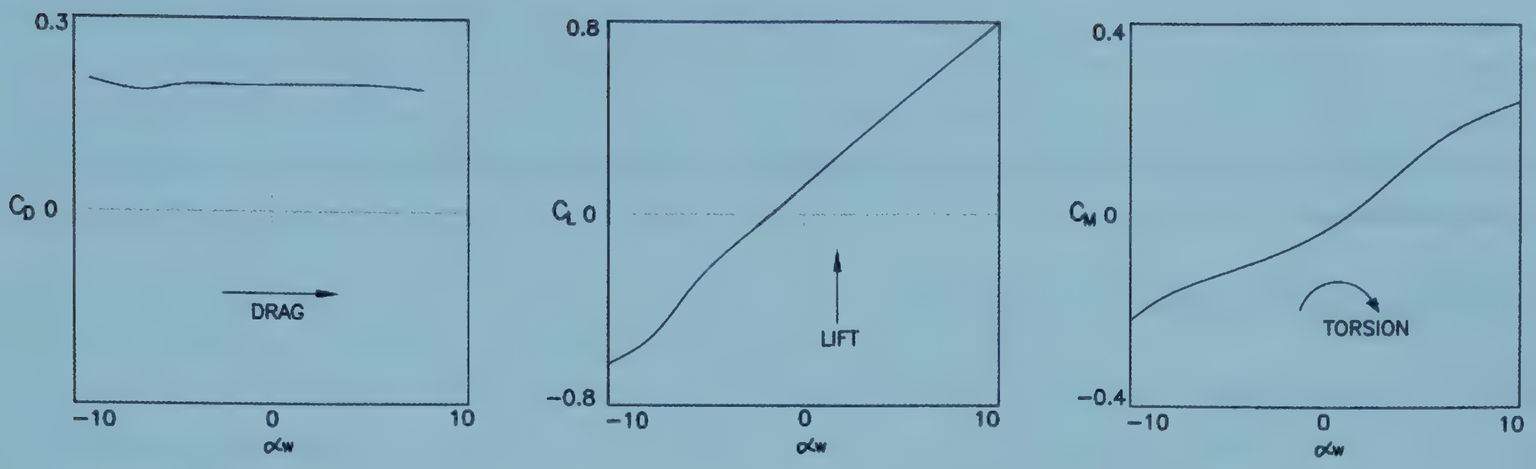


Fig. 9.7 Typical drag, lift and moment coefficients for a bridge deck

For bridges which are not sensitive to dynamic wind effects, the force coefficients (drag and lift) as given in IRC:6 may be selected depending on the cross-sectional shape and aspect ratio (b/D) of bridge deck. However, IRC:6 **[1]** stipulates only drag and lift coefficients of bridge deck cross-section. Aerodynamic force coefficients (aerodynamic drag, lift and moment coefficients at wind angles $+6^0$ to -6^0) of bridge deck shall be obtained through numerical studies or wind tunnel studies.

The utility of mean wind force coefficients or aerodynamic coefficients is not limited in the determination of mean wind load. These coefficients and their derivatives with respect to the angle of attack ($dC_L/d\alpha$) and ($dC_M/d\alpha$) are used to determine buffeting forces and to check the galloping stability.

9.9.5.2 Tower

The wind effects are governed by the structural properties and geometric shape of the tower as well as the characteristics of approaching wind. For hourly wind speed and pressure variation along the height of the tower, turbulence intensity depending in the terrain category at the location of bridge IS:875: Part 3:2015 **[2]** shall be referred. The wind load acting on tower shall be determined for a wind speed at a height 0.65 times height of tower from mean ground/water surface.

The tower may be treated by the quasi-static approach for wind load computation, provided that, the following criteria is satisfied:

a) ratio of height to minimum lateral dimension is less than about 5.0 and

b) natural frequency in the first mode obtained from free vibration analysis is more than 1.0 Hz.

For the computation of along wind forces, the drag coefficient for a square cross-section shall be taken as 2.0 or depending on the H/d ratio and t/b ratio as given in IRC:6 **[1]**.

If the conditions stipulated in a) and b) are not satisfied, the tower shall be prone to buffeting, and Gust factor method as given in IS:875: Part 3:2015 **[2]** shall be used for computation of along wind forces after computing gust factor, considering both background and resonance component and it shall be used in conjunction with hourly mean wind pressure.

Wind load on tower shall be computed at various stages of construction as well as for completed configuration.

Wind load on towers shall take into account dynamic and static wind pressure, buffeting, and vortex shedding.

9.9.5.3 Stay Cable System

Cables in a cable-stayed bridge are subject to substantial forces due to wind and this is transferred to their ends into the deck and the towers. For longer spans, the reaction on the tower may match the direct force it experiences due to wind. Stays in a cable stayed bridge are like strings under tension.

For circular sections, such as for a cable, the force coefficient depends on how the wind flows around it and is dependent upon the wind velocity, kinematic viscosity, diameter of the cable, and surface roughness. In general, the force coefficient for a circular section varies with Reynold's number (R_e) and surface roughness. The dependence of the force coefficient of a circular section on R_e is due to the change in the wake developed behind the body.

The drag force from the wind per unit length of the cable shall be calculated using Eq. 9.10.

$$F_D = \frac{1}{2} \rho V_Z d_c C_D G \qquad \text{(.....Eq.9.10)}$$

where,

ρ - is the density of air (1.20 kg/m^3 for standard temperature and pressure)

V_z - Hourly mean speed of wind in m/s at mean height H of cable as per IRC:6

G - Gust factor depending on the terrain category.

C_D - Drag coefficient for the cable shall be taken as 1.2 if $d_cV_z < 6$ m^2/s and 0.8 if $d_cV_z > 6$ m^2/s.

9.9.6 *Buffeting Analysis*

Buffeting analysis of the cable stayed bridge in frequency domain or time domain shall be carried out, (if the steady state aerodynamic coefficients and their derivatives are obtained through wind tunnel tests or CFD) to calculate the peak amplitudes and modes of vibration at the design wind speed. By performing a three-dimensional buffeting analysis in time domain, considering the geometric non-linearity of cable elements (by considering the modified E value of cables) the relevant buffeting induced forces (Axial force, BM, SF) on bridge deck, tower and cables can be obtained. Wake buffeting or proximity effects shall also be evaluated for twin deck configurations or adjacent bridges.

9.9.7 *Strouhal Number*

Strouhal Number is required to compute the respective critical wind speed for vortex excitation of bridge deck, tower and cable.

In general, the Strouhal number of bridge deck cross-section varies between 0.1 and 0.2. Strouhal number for box girder supported deck of cable stayed bridges varies in the range of 0.1 to 0.15. This shall be verified using numerical modeling or wind tunnel studies when span of bridge is > 250 m and or when the bridge deck is sensitive to vortex induced oscillation as stipulated in Section 9.7 and 9.8.

For circular cables the Strouhal Number shall be taken as 0.2. For square or rectangular cross-section of tower Strouhal number shall be taken as 0.12.

9.9.8 *Flutter Derivatives*

Flutter is a self-feeding aeroelastic phenomenon which is potentially destructive to long-span cable-supported bridges. The aerodynamic forces generated on the bridge deck couple with its motion and, if the energy input by the aerodynamic forces at high wind speeds in an oscillation cycle becomes larger than that dissipated by the mechanical damping of the bridge structural system, the amplitude of vibration will grow. This increasing vibration will then amplify the aerodynamic forces, resulting in continuously growing self-excited forces and self-exciting oscillations. If this situation continues for some time, it can lead to the ultimate limit state of the bridge and ultimately to collapse.

Flutter derivatives represent the characteristics of self-excited aerodynamic forces of a bridge deck cross-section and are the most important parameters used to identify the incipient stage of flutter, with the estimated critical wind speed for onset of flutter using these derivatives. Flutter derivatives are generally determined by means of wind tunnel studies.

Reference **[11]** can be perused for providing detailed information and understanding about the application of flutter derivatives to the study of aerodynamic stability of long span bridges.

9.10 Investigations for Design Parameters

The aerodynamic parameters required for cable stayed bridges shall be obtained through (i) numerical simulation studies and or (ii) wind tunnel studies

9.10.1 *Numerical Simulation/CFD Studies*

Numerical simulation based on computational fluid dynamics (CFD) provides an alternative to as wind tunnel testing, which often proves to be expensive and time-consuming. Also, CFD enables simultaneous determination of force coefficients, pressure distributions, structural response, and flow visualization of structures in various flow fields. This feature in the application of CFD is often helpful and essential for the understanding of the fluid-structure interaction mechanisms that give rise to wind induced vibration.

9.10.2 *Wind Tunnel Studies*

Wind tunnel studies on sectional model shall be carried out for Cable stayed bridge deck if the criteria given in **Table 9.2** is not satisfied and or aerodynamically found sensitive as stipulated in **Section 9.8**, and, also for Cable stayed bridge with main span more than 250 m length.

i) Sectional model tests shall be carried out on as large a scaled model as possible. Sectional model tests shall be performed in both smooth and turbulent flows, covering wind speeds from the lowest practical range, up to and beyond the project specific stability criteria.

ii) Steady-state force and moment coefficients should be measured outside the vortex shedding lock-in range at speed lower than the flutter speed.

iii) Static force and moment coefficients shall be measured for wind angles ranging, as a minimum, between ±6° in increments of 2°.

iv) Drag coefficients shall be normalized with respect to depth of deck (including crash barrier and railings), whereas lift and moment coefficients should be normalized with respect to width (edge to edge) of deck cross-section.

v) Structural damping ratios shall be set in the range of their expected full-scale values when dynamic sectional model is used for checking the aerodynamic susceptibility.

vi) Aerodynamic derivatives used to determine flutter speed analytically shall be extracted from testing at wind speeds beyond the flutter speed criteria. At least 8 derivatives shall be identified for 2 degree-of-freedom (2-dof) tests, and 18 derivatives shall be identified from 3-dof tests.

vii) If a free vibration method is used, aerodynamic derivatives shall be extracted in smooth flow.

viii) A full model should therefore be used only where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations necessitate its use.

ix) The following typical scales for the various types of wind tunnel tests are recommended:

x) Topographic models 1:2000 (To examine the influence of topography or other perturbations)

Aero-elastic models 1:200 to 1:100

Section models (stability or time average coefficients) 1:80 to 1:40

However, for the range of span of cable stayed bridges in focus in these guidelines (upto 500 m), it should commonly not be required to deploy a full aeroelastic model.

Some additional information is given in **Annexure A3**. Though for carrying out wind tunnel model testing of a bridge, essentially, specialist literature, and, preferably also an experienced wind tunnel establishment should be consulted.

9.11 Mitigation Measures

Mitigation measures for aerodynamic design of cable stayed bridge include either or combination of the following modification such as (i) geometric (ii) structural and (iii) mechanical means.**[4]**

One or more of the measures mentioned above to augment the aerodynamic stability of the bridge deck system, have mostly to be based on experience and good practice rather than any theoretical analysis. Essentially, these have to be proved through model testing.

Furthermore, these measures do not necessarily have to be provided throughout the length of the span, and, partial treatment of parts of the span may suffice.

9.11.1 *Deck*

i) The vortex-induced response of a bridge deck can be influenced by a tiny change in the configuration of the leading and trailing edge of the cross-section. Similarly, the installation of the solid handrail / crash barrier on the deck section may increase aerodynamic instability. The horizontal member of the handrail can play an important role in the vortex-induced oscillation.

ii) New Jersey type crash barrier used in cable-stayed bridge decks acts as a “flow tripper” and causes vortex-induced motion. Therefore, it is imperative for designers contemplating the use of this type of crash barrier to perform wind–tunnel tests for confirming the stability of the proposed deck cross-sectional configuration. In fact, 50 to 70% porous steel crash barriers can improve the stability performance of the deck significantly and has been implemented on many bridges around the world.

iii) The two efficient ways to suppress the vortex-induced oscillations are to restrict the formation of a separation bubble very close to the leading edge of the bridge deck or make it spread very far of the wake. Practically, this is achieved with the help of aerodynamic appendages such as the provision of Baffle Plates, Fairings on part of the span, for example shown in **Fig. 9.8 (a) & (b),** to disorganize the wind and disturb the rhythm with which vortices may be shed.

iv) A box girder deck section, symmetrical about the vertical center plane and featuring rounded off edges or use of streamlined box section, with a wind nose is preferable to minimize the effect of vortex-induced oscillation. In case, unfavourable downstream flow develops, it can be compensated by the introduction of guide vanes **Fig. 9.8 (b).** The aerodynamic functions of guide vanes are as follows:

  (a) To reduce or avoid the formation of coherent large-scale vortices in the wake of girder due to relatively abrupt change of surface angle, and thus eliminate vortex-induced vibrations at low speeds.

  (b) To increase the aerodynamic damping in torsion and thus enhance the critical wind speed for vortex shedding.

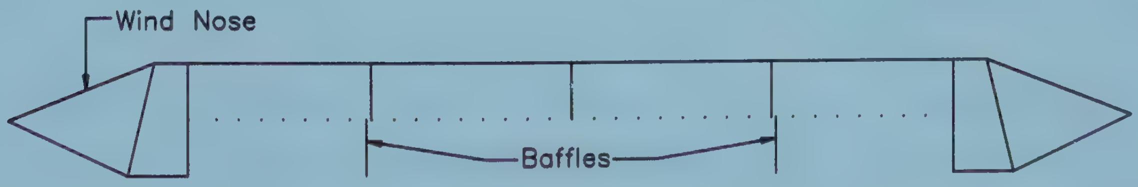


Fig.9.8 (a) Baffle plates

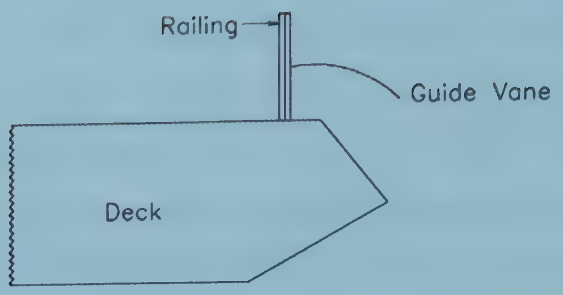


Fig. 9.8 (b) Guide vanes

v) Accumulation of ice or snow may effectively change the aerodynamic shape in the critical areas of bridge deck-like edges and can alter the response of the bridge deck due to vortex-induced oscillation. The presence of movement of traffic on the bridge can also alter the response due to vortex shedding.

vi) In an open–slotted bridge deck by providing a lateral bracing system torsional strength could be enhanced thereby increasing the aerodynamic stability.

vii) The aerodynamic instability in long-span bridges may be reduced, and in some cases eliminated, through the use of proper combination of slots (openings on the bridge deck floor) and fins (lateral projections).This is because the lateral or central slots eliminate vertical instability and outside fins reduce or eliminate torsional instability. A combination of the above two in suitable proportions has been found to have yielded good results for both vertical and torsional instability.

viii) Critical velocity for the flutter of bridge deck with H, Π and flat plate type cross-section can be enhanced by the installation of open grating in the deck. Further, it is better to locate the open grating as close to the edge as possible in the section with an area of opening of more than 30% of the deck area.

ix) Bridge decks with impermeable railing are likely to be prone to a single degree of freedom flutter in torsion. Since railings are required to guard the traffic, they should be made permeable for better aerodynamic stability.

x) Aerodynamic stability of super long-span bridges could be achieved by controlling the stiffness distribution along the span of bridge deck i.e. by flutter mode shape control.

xi) he critical flutter wind speed of long- span flexible bridges could be increased by the installation of tuned mass dampers.

xii) Active control systems are found useful in wind sensitive bridges to enhance the comfort of the bridge users and to reduce fatigue damage.

xiii) The connection between deck and tower, i.e., either a floating deck of elastically supported connection between the deck and tower are appropriate for aerodynamic consideration.

xiv) A damper could be used as a temporary measure to prevent vortex-induced oscillation of a cable-stayed bridge under construction.

9.11.2 *Tower*

The improved aerodynamic design of tower is achieved with the help of corner deflectors, central slit in cross-section as well as with corner modifications **Fig.9.9**.

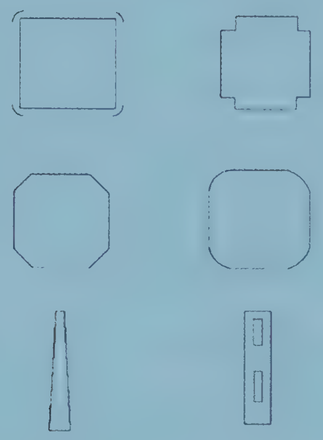


Fig. 9.9 Geometric Modifications /Corner Modifications of Tower

Geometric modification of tower cross-section such as introducing corner-cut is one of the well-known methods to reduce the vortex-induced vibration of tower with rectangular sections **[12]**

Wind-induced vibration during erection affects the safety and serviceability of the structure; typical examples are pylons in the free-standing stage. In tall pylons, not only the first fundamental mode but also higher modes need to be checked and controlled, if needed.

Tall steel pylons require temporal control for wind-induced vibrations In such situations, mechanical control such as sliding-block-type dampers were installed (by connecting to the top of pylons by wires) to control the vortex-induced vibration of pylons of bridge during the erection. Also, applications can be found in long-span cable-supported bridges including Yokohama Bay Bridge, in which viscous dampers were employed. Though TMD and TLD are useful in controlling vibrations, they are effective only in small band of frequency **[13]**. TMD was also used to control the cantilevered girder of a cable-stayed bridge during erection. By considering cost, maintenance and reliability aspects passive TMD's are preferred instead of active control systems.

Furthermore, to alleviate the wind-induced oscillatory problems of the tower during operation stage, Tuned Mass Dampers (TMD) could be installed.

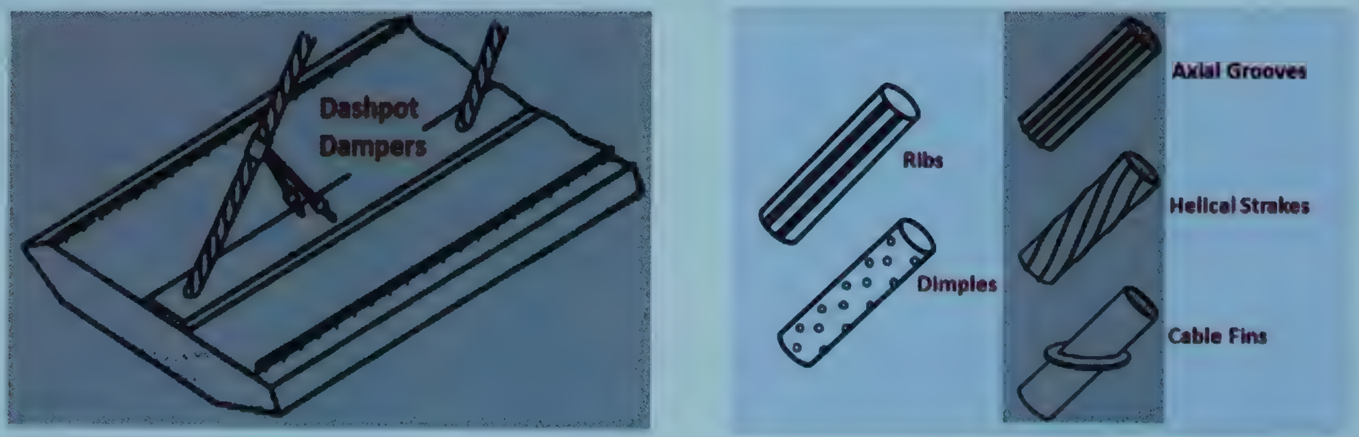
9.11.3 *Cables*

i) In case of cables which are found to be susceptible to rain-wind induced oscillation, surface modification of HDPE duct could be adopted. Surface modifications, **Fig. 9.10** such as double helical rib or incorporation of dimples on the HDPE duct keeps the stay cable stable by preventing the occurrence of the water rivulet under certain wind conditions. This is a preferable solution as it does not increase the drag of the cable while maintaining the required aesthetical appearance of the stays. However, its efficacy will have to be tested by wind tunnel testing, unless the information is found in specialist literature.

ii) Structural control of vibrations by interconnecting cable-ties has been used very frequently, either temporarily, before the installation of damping devices **[3]** on the cables, or as a permanent measure.

iii) Cross ties are recommended when the vibration frequencies of the deck or tower are close to the frequencies of the stay cables. They consist of a bundle of stainless-steel wire ropes fully embedded in HDPE and other dampening plastic material. They are to be tested for fatigue and damping capacity. A steel collar provides the fixation of the ropes on each stay and facilities for any orientation.

In terms of structural behaviour, the addition of cross-ties to the stay cable system creates intermediate supports at those elements and, consequently, increases their natural frequencies for vertical vibrations. With a proper design of these ropes, the fundamental frequency of the stay cables is shifted to relatively high values, thus reducing the possibility of internal resonance with global bridge modes at the fundamental mode shapes. Another effect of the installation of cross-cables is an increase of the damping **(Table 9.6)**.



(a) Dampers Installed on the Stay

(b) Surface Treatment to Control Rain-Wind Oscillations

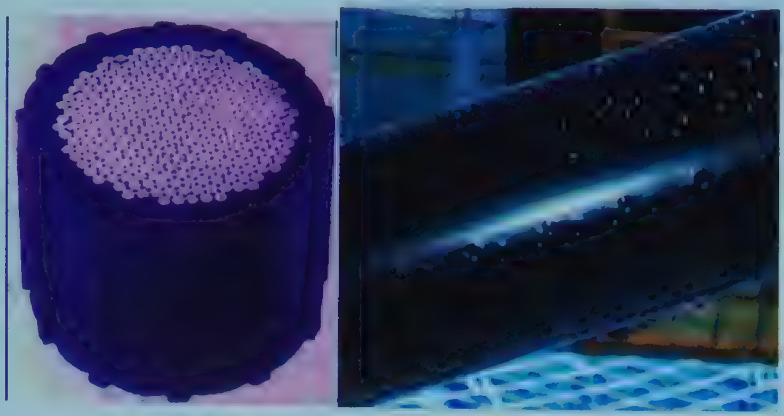


Fig. 9.10 Illustrating Measures for Controlling Stay Oscillations in Stay cable

Table 9.6 indicates the remedial measures used in some of the existing cable stayed bridges.

Table 9.6 Stay Cable Vibration Mitigation Measures

Sl no.	Bridge Name	Year of construction	Main Span (m)	Cable vibration observed	Oscillatory amplitude (m)	Mitigation measure adopted
1	Kohlbrandt, Germany	1974	326		1.0	Viscous dampers
2	Tjorn, Sweden	1981	360	Vibration during light rain	-	Viscous dampers
3	Brotonne, France	1977	320	Vibrating in 15m/s wind	0.6	Viscous dampers
4	Ben-Ahin Belgium	1987	168	Vibration noted during light drizzle and 10 m/s winds	1.0	Petroleum wax grout added in the guide pipe and cross cables
5	Yobuko	1988	250	Vibration during light rain	0.15	Manila ropes attached to the cables
6	Aratsu Japan	1988	186	Vibration during light rain	0.6	Viscous dampers
7	Wandre. Belgium	1989	168	Vibration during light rain and 10m/s winds	0.5	Petroleum wax fill in the guide pipe
8	Alzette, Luxembourg	1993	130	Vibration during drizzle and light wind	-	Neoprene guides inside the guide pipe at deck and petroleum wax fill in the guide pipe
9	Weirton Steubenville USA	1990	250	Vibrations noted when winds are parallel to deck	0.6	Visco-elastic dampers installed in the guide pipe at deck level
10	Tempozan	1990	351	Vibration during rain and 10m/s winds	2.0	Viscous dampers
11	Helgeland Norway	1991	427	Large cable vibrations; depending on deck motion	0.6	Cross-cables
12	Normandie France	1995	853	Cable vibrations during steady 1 to 2m/s winds	1.5	Viscous dampers
13	Second Severn	1997	457	Cable vibration with and without rain	1.5	Cross cables
14	Meiko Nishi	1997	404	Vibration during light rain and low winds	0.55	Cross cables
15	Anzac Australia	1995	345	Vibration during light rain and wind	1.5	Dampers/ surface treatment of cables

9.11.4 *Use of dampers*

Use of dampers is unlikey to be there for cables up to 60 m length. For cables above 60 m length additional damping requirement can be worked out using Scruton number (Equation 9.7).

Both external or internal dampers could be adopted to enhance the damping ratio of the cables. External damper may be hydraulic damper and are visible and requires maintenance. Hence, not preferred in comparison to internal dampers which are not visible and maintenance free.

Internal dampers are of two types- (i) Internal Elastomeric type (IED) generally used for cables up to 150 m length and for cables with more than 150 m length (ii) Internal Hydraulic dampers (IHD) or a combination of IED with IHD may be used.

Internal Elastomeric Damper (IED)

A cast piece of special rubber with high damping characteristics and a specially designed shape is placed between the stay cable and the guide tube (deck and pylon). If the damping characteristics need to be increased, viscous oil can be placed in a special chamber. This is a very compact system with virtually no maintenance. It can provide sufficient damping for cables as long as 150 m. This damper is completely invisible from the deck. For details **Chapter 11** shall be referred.

Internal Hydraulic Damper (IHD)

Internal Hydraulic Damper (IHD) is generally used for cables longer than 150 m. It may be used in parallel with the IED, to increase the damping characteristics of the whole system. It has the same advantage as that of IED, i.e., it is not visible and almost maintenance free.

Limiting Value of Cable Vibration Amplitude-Comfort Criteria

Study on sensitivity of bridge users to cable vibration has indicated the comfort criteria for cable displacement can be described as follows for cables with frequency in the range 0.5 to 2 Hz. The cable displacement shall be kept below one d_c (d_c is the cable diameter) and in no case shall exceed 2.0 d_c **[10]**.

Wind Related Terms & Notation

1. **Bluff body:** A bluff body can be defined as a body that as a result of its cross-sectional shape has a separated flow over a substantial portion of its surface, resulting in the formation of a wake having significant lateral dimensions and unsteady velocity fields.
2. **Turbulence:** Turbulence in wind originates from the action of shear stress at ground level due to surface roughness and ground obstructions, which are transmitted upwards through the atmosphere leading to the formation and decay of eddies.

3. **Fairings:** Fairings are appendages of triangular or circular elements attached externally at the leading edges of a bluff bridge deck to enhance their aerodynamic performance, especially concerning vortex-induced oscillation, by altering the flow pattern around it.

4. **Guide Vanes:** Guide vanes or deflectors are provided at the corners of the bluff body (bridge deck or pylon) to modify the wind flow pattern by preventing flow separation at sharp corners, thereby mitigating vortex-induced vibration.

NOTATIONS

Acronyms used are defined wherever these occur in the text and have been collected here for convenience.

V_{re}	Reduced Velocity
V	Velocity
B	A typical dimension, width of deck
S	Strouhal number
S_c	Scruton number
f	Frequency of Vibration, vortex shedding; subscript c,p pertain to cable, pylon respectively
D	A typical dimension for the cross section, subscript p pertains to pylon
d_c	Diameter of cable
F_L	Force due to lift
F_D	Force due to drag
F_M	Twisting moment
d	Depth of deck
L	Length of span
C_D	Coefficient of drag for a deck cross-section
C_L	Coefficient of lift for a deck cross-section
C_M	Coefficient of moment for a deck cross-section
α, α_w	Angle of incidence of wind
K_α	A torsional constant for the deck
f_t	First torsional mode frequency of bridge deck
f_v	First vertical mode frequency of bridge deck
I_u	Turbulence intensity
V_b	Basic Wind velocity
V_d	Design wind velocity

V_f	Critical wind velocity for classical flutter
V_{Rf}	Reduced critical wind speed
m	Mass per unit length of the element concerned
r	Polar radius of gyration of effective bridge deck cross-section at the centre of main-span
V_{crf}	Critical wind speed for initiation of single degree of freedom flutter
R_e	Reynolds number
h_a	The vertical vortex induced oscillatory amplitude
ζ_s	Critical damping ratio of bridge deck
β_{ds}	The shape correction coefficient
β_t	A coefficient, =1
Z	Reference altitude of deck, cable
z_0	Surface roughness
δ	Logarithmic decrement of damping
P_b	Aerodynamic susceptibility parameter
Sw	Mass damping parameter
k_1	Velocity conversion factor
V_H	Mean hourly wind velocity
G	Gust factor
ρ	Unit weight per unit length

9.12 Additional recommended reading is given under reference [14] of this Chapter.

CHAPTER - 10

STAY CABLE SYSTEM, MATERIALS AND QUALITY ASSURANCE

10.1 General Requirements

10.1.1 *Principles for Cable Stay Protection*

The basic principle for a cable stay protection means that the cable stays have to be protected against fretting and corrosion of the tensile elements so that long service life is guaranteed before replacement becomes necessary. Information in International Guidelines **[1], [2], [3],** are shown in **Annexure 4**.

Design life of stay cables is always lesser than service life of structure. Considering the difficulties in replacement of stay cables, it is necessary that stay cables require to be replaced only once in the entire service life of structure. Appropriate maintenance is considered to be in order during the period before the stay has to be replaced.

10.1.1.1 Fretting –Resistance Precautions

The main tensile elements of which cable stays are made may be in contact with each other and with other steel parts which may occur in the free length (contact between individual wires) and in anchorage zones or zones of attachment to the stay or saddle zones. As a result of microscopic and frequent movements at these contacts, there is loss of material by wearing and removing oxides thus formed in powder form. The effects of these phenomena are called as wear (called fretting corrosion) and fatigue cracking (called fretting fatigue) which must be prevented by providing an interface material which either mechanically isolates or lubricates the contact between the steel parts in all zones of cables.

10.1.1.2 Redundancy of Corrosion –Protection Barriers

Corrosion protection of the steel tensile elements of cable stays must be provided with a degree of redundancy by means of two complementary **Fig. 10.1**, nested barriers:

- The first barrier (or internal barrier) is a protective coating applied directly to the main tensile element, and must cover the full length of the cable in all zones without a break.
- The second barrier (or external barrier) consists of an outer casing and an intermediate medium between the outer casing and the internal barrier. To prevent the internal barrier being consumed, the outer casing must be completely airtight and watertight, in both the free length and the anchorage zone. In addition, the intermediate medium must prevent any water or moisture that might get inside the external barrier from reaching the internal barrier.

Any damage caused to second barrier should not expose the main tensile elements directly to environmental aggression, thereby justifying the requirement of the internal barrier.

10.1.1.3 Continuity of Corrosion-Protection Barriers

The two corrosion-protection barriers as described above must be continuous throughout the length of the stay, and especially in the anchorage zone.

10.1.2 *Internal barrier*

Sacrificial metal coating layer such as a galvanized coating is considered as the best internal barrier.

In most cases, the internal barrier must meet the dual requirements of corrosion protection and protection against fretting. Experience with galvanization demonstrates the favourable role of zinc not only as sacrificial protection against chemical corrosion but also as a contact interface whose wear-induced particles do not present the same drawbacks as rust from steel-on steel contact.

10.1.2.1 Protection by Metal Coating

The quality and lifetime of this internal protection depends on the type and thickness of the metal coating.

The following criteria are to be considered when choosing the thickness of the metal coating:

- provide sufficient thickness to prolong the lifetime of the sacrificial protection
- comply with the technological limits of the interface between the steel and the protective metal coating (adherence, continuity, ductility, etc.)
- for stays consisting of strands anchored with split-cone wedges, avoid excessive coating thickness as too much zinc could clog the teeth of the jaws when the strand is pulled up using stressing equipment
- for multi-layer strands with Z-shaped wires (locked-coil cables), comply with the dimensional tolerances that will ensure good wire interlock;
- for multi-layer strands with round wires, ensure that the metal coating does not compromise the strength of the wires

The protective metal coating must be applied to individual wires as they are manufactured.

10.1.2.2 Non-metallic Protection Equivalent to Galvanization

Instead of a galvanized metal coating, a protective coating using new materials (such as epoxy coating) may be used, subject to it being demonstrated that the coating has a level of performance equivalent to that of galvanization in respect of the following properties:

- continuity and adherence to the tension element under static (bending) and dynamic action effects in service;
- lubrication of contact points to prevent fretting corrosion and fatigue;
- corrosion protection.

10.1.3 *External Barrier*

The external barrier can take a variety of forms, in accordance with the type of cable stay and special requirements specific to the structure. It is generally designed and erected by the proprietor of the stay system employed, who must define the following in his technical literatures:

- The type of external barrier for the free length of the cable stay (outer casing and intermediary medium);
- The arrangements for ensuring the continuity of the barrier (intermediary medium) throughout the anchorage zone.

10.1.3.1 Watertight Outer Casing

For the free length of the cable stay, the outer casing is usually a sheath of one of the following kinds:

- stay pipe made from plastics tube: polyolefins, High-Density Polyethylene (HDPE), or similar
- plastics sheath extruded directly onto individual strands, in the case of individually sheathed multi-strand cable stays
- steel stay pipe made from pipe sections welded together; these are either protected against corrosion or are made of stainless steel.

Casing, paint or epoxy coating of multi-layer strand cable stays can give adequate protection, however it does require demanding periodic maintenance.

When the external corrosion-protection barrier is a stay pipe, it can also fulfil other functions, particularly:

- Mechanical functions (limiting cable-stay drag or rattling of main tensile elements);
- Aerodynamic functions (limiting rain & wind induced instability);
- Aesthetic functions (colour).

10.1.3.2 Intermediate Medium

In the free length of the cable stay, the intermediate medium is generally:

- either a blocking medium in the annular space, using a suitable infilling product (generally wax, grease, or resin);
- or a permanent controlled-humidity air flow around the main tensile elements, using a cable dehumidification protection system. Such a system prevents condensation between the two barriers.

10.2 Stay Cable Types

In earlier days of construction of cable stayed bridges, parallel strand system with Outer HDPE pipe injected with cement grout (as an intermediate medium) were used. But, this proves to be a failure for following reasons:-

- Cement grout cracks due to shrinkage and bending of stay cables.
- Difference between the coefficient of expansion of the cement grout and HDPE was source of disorder in stay pipes injected with cement grout.
- Between interface of cement grout and main tensile steel, fretting corrosion was observed. Infact, fatigue tests performed have revealed traces of iron oxide at main tensile steel -grout interface.

  Currently, stay cables are made up of different kinds of tensile steel elements and can be broadly grouped in following four categories based on strand system or wire system.

  - Parallel Strand System-PSS
  - New Parallel Wire System-NPWS
  - Locked Coil System (LCS)
  - Epoxy Coated And filled Strand (ECF strand)

  In the majority of recently constructed cable stayed bridges, Parallel Strand Cable (PSS) has been used.

10.2.1 *Parallel Strand System (PSS)* ***(Fig 10.1)***

The main tensile elements of PSS Cables are group of strands, wherein each strand is made up of seven ply wires. Each wire is of 5 mm in diameter except the central wire which is of larger diameter than 5 mm. Tensile strengths of such strands available in market are, 1860 MPa for 15.2 mm and 1770 MPa / 1860 MPa for 15.7 mm dia **[6]**. However, 5 mm wires with higher tensile strength of 2000 MPa have been also developed.

Each wire of a strand is galvanized and such galvanised strands are extruded in individually sheathed HDPE tube & injected with wax or grease.

All such individually sheathed strands are enclosed in an outer sheath or pipe.

Generally, diameter of outer HDPE duct is being governed by numbers of strand grouped in it. Required diameter of the duct for PSS cable is more than other two stay cable types (PWS or MLS). Large diameter of outer HDPE results in more wind loading on structures which is very vital criteria for large span. In last decade, stay cable supplier's have developed new PSS cable system of threading of strands, wherein strands can be accommodated in more compact manner and thus reducing the void-ratio inside outer duct.

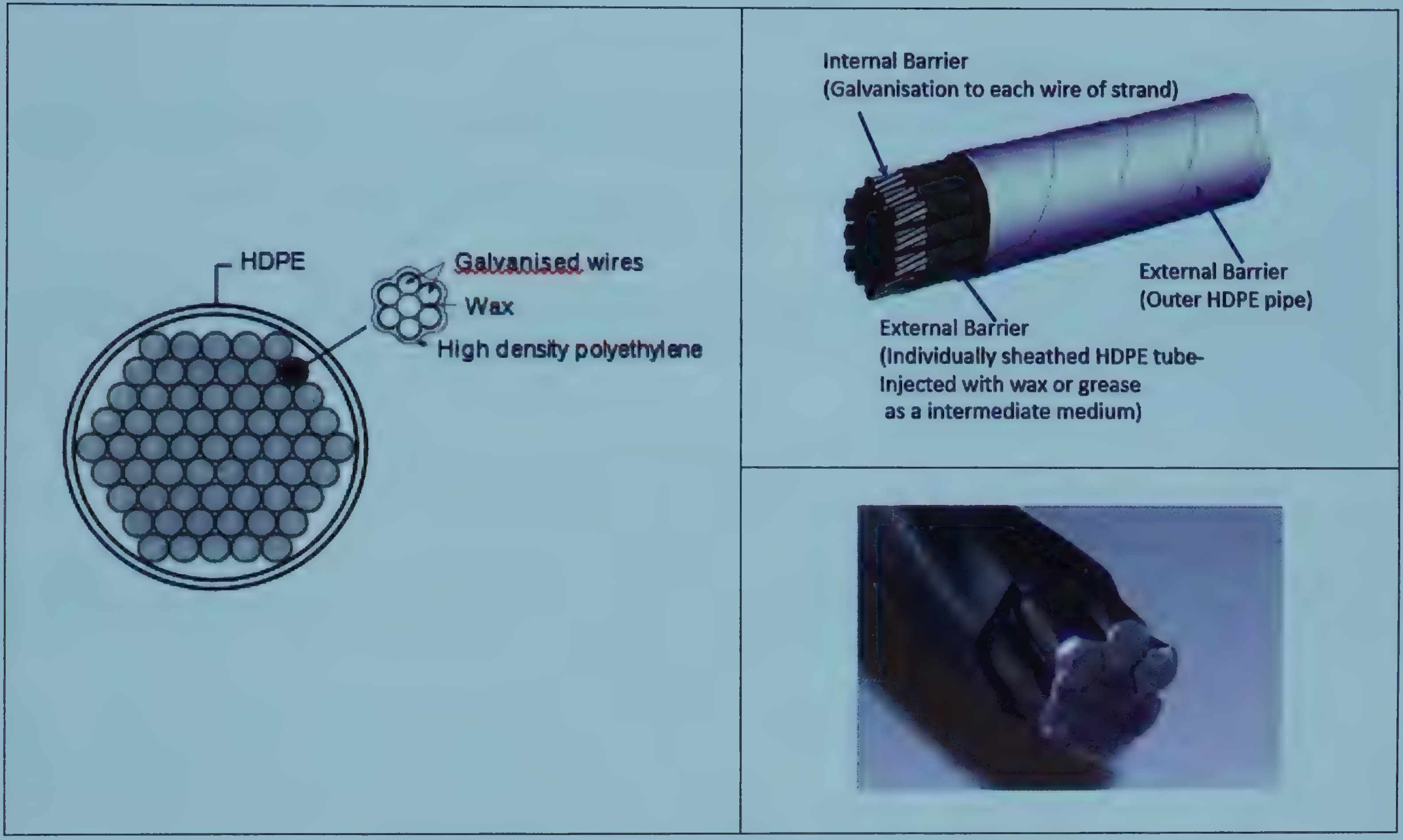

HDPE
Galvanised wires
Wax
High density polyethylene
Internal Barrier
(Galvanisation to each wire of strand)
External Barrier
(Outer HDPE pipe)
External Barrier
(Individually sheathed HDPE tube-
Injected with wax or grease
as a intermediate medium)

Fig. 10.1 Cross section of Parallel Strand System (PSS) cable

A cable with 156 nos of 15.7 mm strand having a max breaking load of 41.3 MN has been used in Maumee River Crossing in USA.

10.2.2 *New Parallel Wire System (NPWS)* ***(Fig. 10.2)***

NPWS cables are a development of PWS cables, which was developed and used in Japan in the 1970s and 80s. They are composed of a wire bundle. For the NPWS cables, wires are slightly twisted with a long lay to ease the reeling and unreeling and make the cable self-compacting when subjected to axial tension. Besides the slight twist the NPWS cables are also characterised by having the protecting outer polyethylene cover extruded directly onto the wire bundle so that no void will exist between the wires and the surrounding pipe.

The wires used for NPWS cables are of 7 mm diameter with tensile strengths of 1570 MPa – 1770 MPa.

The wires are protected by galvanisation and the wire bundle is wrapped in filament tape as well as the extruded high density polyethylene (HDPE) outer casing.

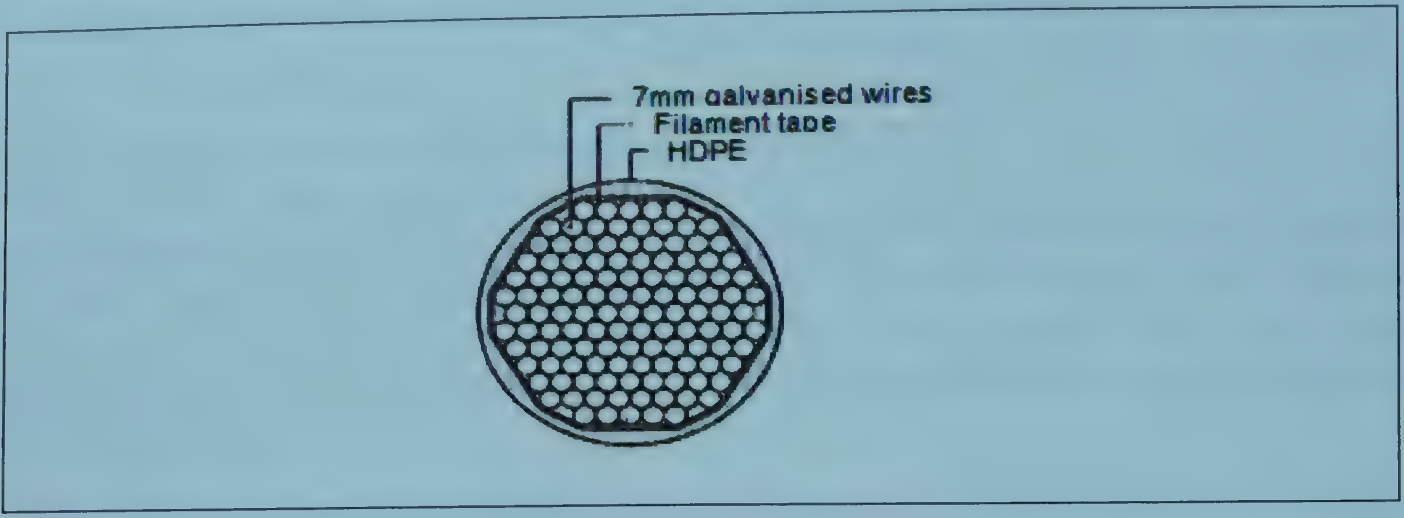


Fig. 10.2 Cross section of New Parallel Wire System (NPWS) cable

NPWS cables are fabricated in sizes ranging from 19 nos (7 mm wires) to 499 nos (7 mm wires).

Diameter of whole cable ranges from 45 mm to 192 mm and the maximum breaking load for a 192 mm NPWS cable is 33.99 MN based on 1770 MPa tensile strength.

10.2.3 *Locked Coil System (LCS)* ***(Fig. 10.3)***

Locked coil cables are composed of two types of twisted wire.

In the core of cable, normal rounded wires are wound helically round a core wire. Such layers in inner core are wound in alternate directions.

In outer layer, wire are of a special Z-shape (2 to 4 layers of interlocked wires). The Z- shape is so chosen so that wires interlock and ensures a tight surface. Also the fit of the Z-wires leads to a very small void ratio inside the cables. The wires used for LCS cables generally have tensile strengths of 1370 MPa to 1570 MPa.

In all cases, all wires shall be galvanised. In most cases, this corrosion protection is only combined with the traditional surface treatment of painting. If required a polyethylene tube can be extruded onto the LCS cables in a similar manner as employed for NPWS cable. However, inspection and repair of painted cable is easier.

Such cables are manufactured in full length and full cross-section and delivered with sockets on large reels.

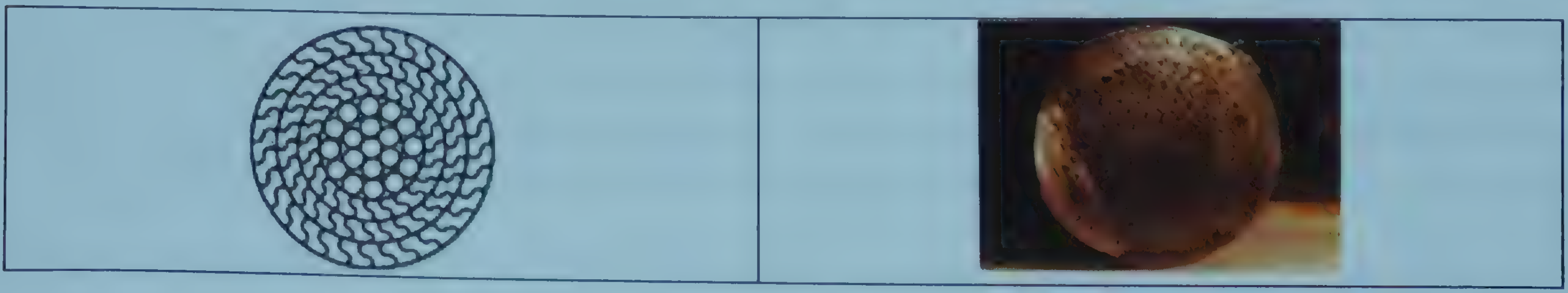


Fig. 10.3 Cross Section of a Locked Coil System (LCS) Cable

10.2.4 *Epoxy Coated and filled Strand (ECF strand)* ***(Fig. 10.4)***

Filled epoxy coated strands are available for seven wire assemblies **[7]**. The strands are coated with epoxy resin which is fully filled among each of the wires, giving its high corrosion protection and outstanding resistibility against fretting fatigue. Since, epoxy resin is not suitable for use in prolonged UV light radiation environments, PE sheathed ECF strand are recommended.

However, great care has to be exercised to prevent abrasion and damage to coating during its transportation and site installation. The most basic precaution needed is to prevent ECF strand from direct contact with something harder than the epoxy resin like metal or concrete.

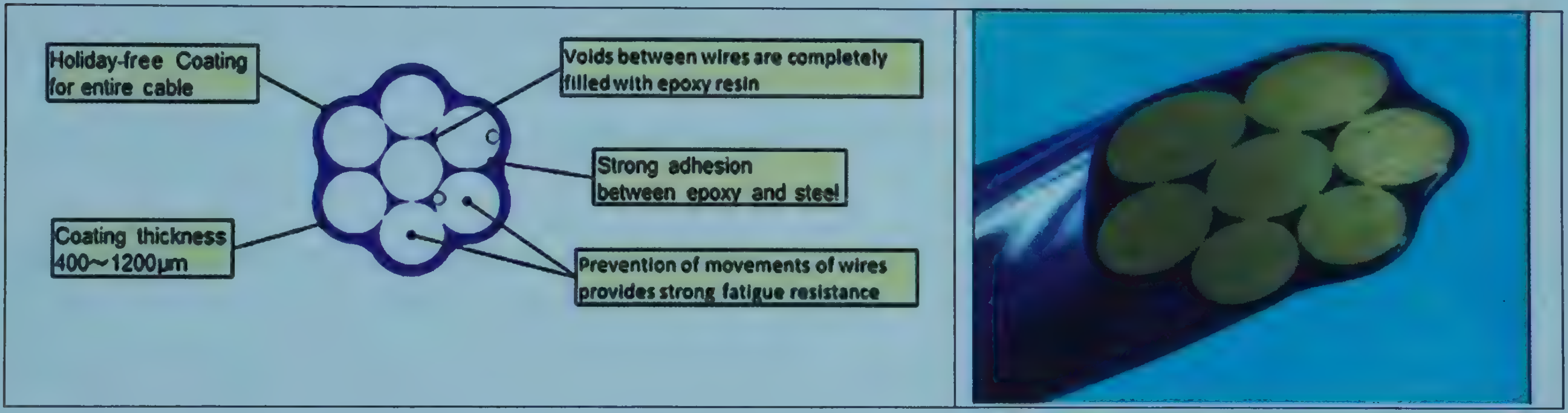

Holiday-free Coating for entire cable
Voids between wires are completely filled with epoxy resin
Strong adhesion between epoxy and steel
Coating thickness 400~1200μm
Prevention of movements of wires provides strong fatigue resistance

Fig. 10.4 Cross-section of Epoxy Coated and Filled Strands

Special wedges that bite through the epoxy coating and grip the prestressing steel are required. Wide & deep teeth are needed for good anchorages of the strand.

10.3 Qualification Criteria and Testing of Stay Cables System

Three different levels of testing of stay cables are recommended **[2]**:

- Initial approval (qualification) testing of the stay cable system
- Suitability testing of the stay cable system for a particular project
- Quality control testing of the stay cable components for a particular project

The cable stay has three parts, viz. the lower end anchorage, the upper end anchorage or possibly saddle, and the MTE (Main Tensile Elements). All these elements are influenced by the variation of stresses in the strands. As the length is long, the cable stays are more sensitive in cable stayed bridge than in a extradosed bridge, to fatigue effects and also to dynamic vibrations which also induce fatigue effects. Moreover, in the majority of cable stayed bridges, the stress variations are more significant than in a Extradosed Bridges.

In view of this, the qualification and testing criteria for cable stay bridges is more stringent compared to that of extradosed cable system. Upper Stress and fluctuating stress range for testing of cables are different for cable stay bridges and extradosed bridges **[2]**.

If for a particular project all the cables have $\Delta\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) less than 50 MPa, the qualification testing (initial and suitability testing) requirements of the extradosed bridges shall be adopted else all the requirements of the cable stayed bridges shall be adopted (**Annexure-A4**).

10.3.1 *Laboratory and Independent Witnesses*

Initial approval testing or suitability testing shall be executed in a laboratory which fulfils the requirements provided in EN ISO/IEC17025 or equivalent standard. In addition following requirements shall be considered:-

- The laboratory shall be independent and have enough international experience with testing of cable systems.
- The testing machines and measuring equipment's shall be calibrated according to ISO 7500 or equivalent standards. The calibration certificates shall be included in the test report. The uncertainty of the values measured with the measuring equipment shall be within +/-1.0%.
- The laboratory shall provide automatic system to detect failures of tensile elements during fatigue testing.
- The laboratory shall observe all safety measures for safety of their workers as well as the machines.

Suitability testing can be also performed in private facilities which fulfil the same requirement as independent laboratories provided that critical testing steps such as installation of the test specimen , implementation of testing parameters, tensile tests or friction tests (wherever applicable for saddle) and reporting are witnessed by an independent body or expert , both to be approved by the owner.

All test procedures, methodology and acceptance criteria of test shall be submitted by supplier to owner prior to the test and the same shall be approved by owner.

10.3.2 *Initial Approval Testing (System Qualification testing)*

The objective of initial approval testing of stay cable systems is to demonstrate the feasibility (practicability) and performance (reliability) of a proposed stay cable system design comprising a range of stay cable sizes and using the proposed materials.

Stay cable system supplier tests (conducted in last 5 years) are considered as valid as long as there are no changes in relevant stay cable details.

For initial testing approval of anchorage/saddle axial fatigue and tensile test, stress variation ($\Delta\sigma_{test}$) range of 200 MPa with upper range of 0.45 f_{GUTS}, with two million load cycles shall be adopted, if all the cables in the project have $\Delta\sigma_{sls}$ more than 50 MPa. Otherwise, stress variation ($\Delta\sigma_{test}$) range of 140 MPa with upper stress of 0.55 f_{GUTS}, with two million load cycles shall be adopted, if all the cables in the project have $\Delta\sigma_{sls}$ less than or equal to 50 MPa.

Initial approval testing of stay cables anchorage assembly shall include :-

- Three Anchorages to be tested for axial fatigue and tensile tests performed on typical sizes (one small, one medium, one large) **[2]**
- Three Anchorage bending fatigue and tensile tests performed on typical sizes (one small, one medium, one large) **[2]**
- One leak tightness test on a small cable size (after the fatigue test) **[2]**

Similar initial approval testing for saddle system (if adopted in project) with isolated stay cable shall include :-

- One complete fully assembled stay cable specimen with saddle setup for saddle axial fatigue, friction and tensile test **[2]**
- One complete fully assembled stay cable specimen with saddle setup for saddle bending fatigue and subsequent tensile test **[2]**
- Effective Saddle friction Coefficient test **[2]**
- One complete saddle leak tightness test **[2]**
- Once a particular extradosed cable system supplier has performed and qualified all acceptable limits of tests as per relevant code of testing for extradosed cable / saddle (if adopted in project) has presented test results for approval for incorporation of the system in the project, it shall be deemed to have been approved subject to the condition that all required tests during suitability testing shall also be satisfied.

But, any change in material specification, structural details or leak tightness details or corrosion protection barriers details will require re-testing of fatigue and tensile tests or leak tightness tests by supplier for initial approval.

If the supplier of a particular component is changed with the same material specification and structural details, only suitability testing is recommended to be done.

10.3.3 *Suitability Testing*

Basic materials, components and detailing of the stay cable system such as strands, pipe and anchorage components shall satisfy the same specifications as the components used for the approval testing. However, they may be fabricated by different suppliers or process than those used during initial approval testing.

The objective of suitability testing is to demonstrate the satisfactory performance of these materials and components of a cable stay system fabricated by a specific supplier and /or a process and intended for use on a particular project.

For suitability testing approval of anchorage/saddle axial fatigue and tensile test, stress variation ($\Delta\sigma_{test}$) range of 250 MPa with upper range of 0.45 f_{GUTS}, with two million load cycles shall be adopted, if all the cables of project have $\Delta\sigma_{sls}$ more than 50 MPa. Otherwise, stress variation ($\Delta\sigma_{test}$) range of 180 MPa with upper stress of 0.55 f_{GUTS}, with two million load cycles shall be adopted, if all the cables of project have $\Delta\sigma_{sls}$ less than or equal to 50 MPa.

Suitability testing of a stay cable system is recommended to be done for each project (based on size of cable being used in project) which include :-

- A minimum of three tensile tests on single tensile elements (strand alone, sample chosen from the source supplier of strand for the project) per batch using laboratory wedges to asses actual strength of the strands (AUTS) **[2]**.

- One stay-anchorage assembly, with wedges and cables using the same system as intended to be used in the project for axial fatigue and tensile test for the size of the cable to be used in the project. If the variation in the sizes of cables is large in a given project then, three tests may be performed instead of one, one with smaller size, second with medium size and another with larger size) **[2]**.

Similar suitability testing of cable system using saddle (if adopted in project) is required to be performed which include :-

- Stay-Saddle fatigue and tensile tests using saddle similar to as carried out for stay-anchorage axial fatigue and tensile test **[2]**.
- One friction tests per project **[2]**.

For each test, 3 samples shall be selected.

If the suitability test does not satisfy the acceptance criteria for a particular material batch, the cable supplier has the choice between the following two options:-

- Replace such batch of material and repeat suitability test associated to the new material batch.
- Do not replace such batch of material and perform two more tests. If either of the two additional tests fails, the batch of material represented by the three samples shall be rejected.

This testing is time-consuming and should be initiated at the early stages of the project to avoid delays. All the testing procedure must be first submitted and approved by the Engineer. After the testing procedure is approved, testing may start, subject to the availability of the special laboratory equipment.

Above tests shall be carried out in a sequence as mentioned above. Failure of any of the above test, may require re-testing of all previous tests.

Stay cable anchorage components have to be installed, while casting the segments. It is, therefore, preferable to have the testing completed prior to casting stay cable segments.

Apart from above, other material checks on strands, PE sheathed duct, filler material and HDPE duct, wedges and anchorages parts shall be also carried out **[2]**.

10.3.4 *Quality Control Testing*

The objective of quality control testing is to determine that the properties of materials and components installed in the cable stayed structure are equivalent to the properties of the materials which were used for initial approval and suitability testing.

Based on agreement between supplier and client for lot supply and test frequency, all / some of tests as specified in **Annexure A4** for testing of various material checks of strands, sheathed duct, filler material and Outer HDPE duct, wedges and anchorages parts are required to be performed **[2]**.

10.4 Cable Vibrations and Control

10.4.1 *General*

The issues related to Cable Vibrations and Control have been addressed in **Chapter 9** dealing generally with Wind Effects and Aerodynamic Design Considerations. This is dealt with here briefly, more particularly to bring out control measures to be applied to stay cables.

As mentioned in **Chapter 9** also, vibrations may occur in stays due to vortex shedding, wake effects, buffeting and cable galloping. It is generally necessary to introduce additional damping in the cables to a level of 3 to 4% logarithmic decrement to suppress such vibrations. Two different types of damping systems are normally considered: internal dampers, installed in connection with the cable anchorage and external dampers, installed independently of the anchorage and fixed to the deck.

The use of internal dampers requires a stiff support at the damper position. For maximum efficiency the internal damper should be placed at a distance of at least 1.5% of the cable length from the anchorage. For a 500 m long cable this means a distance of 7.5 m between the damper and the anchorage point. This is normally best achieved if the cable is taken through the deck and anchored inside or below the deck with the damper placed above the deck and inside the stiff anchorage tube. Suppliers of PSS and NPWS cables have developed internal dampers of various types to suit their cables and anchorage layout. Measurements on several bridges have proved the effectiveness of these dampers. For LCS cables specific internal damper systems have, to the best our knowledge, not been developed.

The design of the cable system should allow for future installations of external dampers i.e. the designer should make sure that in case additional damping is required to suppress unexpected vibrations then a solution is at hand to mitigate them.

10.4.2 *Parametric Excitation*

Vibrations of the cables may also be introduced through structural vibrations at their anchorages either at deck level or tower level. These vibrations can be produced by different actions such as vortex shedding on deck or tower, buffeting or movement of traffic. If these forced vibrations of the anchorages reach a frequency, which coincide with a cable mode, then very large cable amplitudes can occur in the cable. This phenomenon is called parametric excitation. While damping can affect the peak amplitude reached by the cable, it is important to note that adding damping cannot prevent the excitation to take place. It may thus be more prudent to suppress the source.

10.4.3 *Rain-Wind Induced Vibrations*

It is common for stay cables to be subjected to rain-wind induced vibrations, as brought out in **Sec. 9.7** (**Chapter 9**). Both PSS and NPWS cables are prone to rain-wind induced vibrations. The diameter of sheaths is irrelevant but their surface condition plays an important role. Such vibrations are observed on painted steel sheaths, stainless steel sheaths and HDPE sheaths. If the surface of the sheath is very smooth, the top rivulet cannot form. With time, however, the smoothness of the surface reduces due to deposition of dust and the situation changes.

10.4.4 *Dampers and Cable surface Treatment to Control Vibrations* ***(Fig. 10.5)***

Mitigating countermeasures which are similar for both types of cables, and, consist of surface treatment to the outer HDPE tube, are necessarily required to be undertaken.

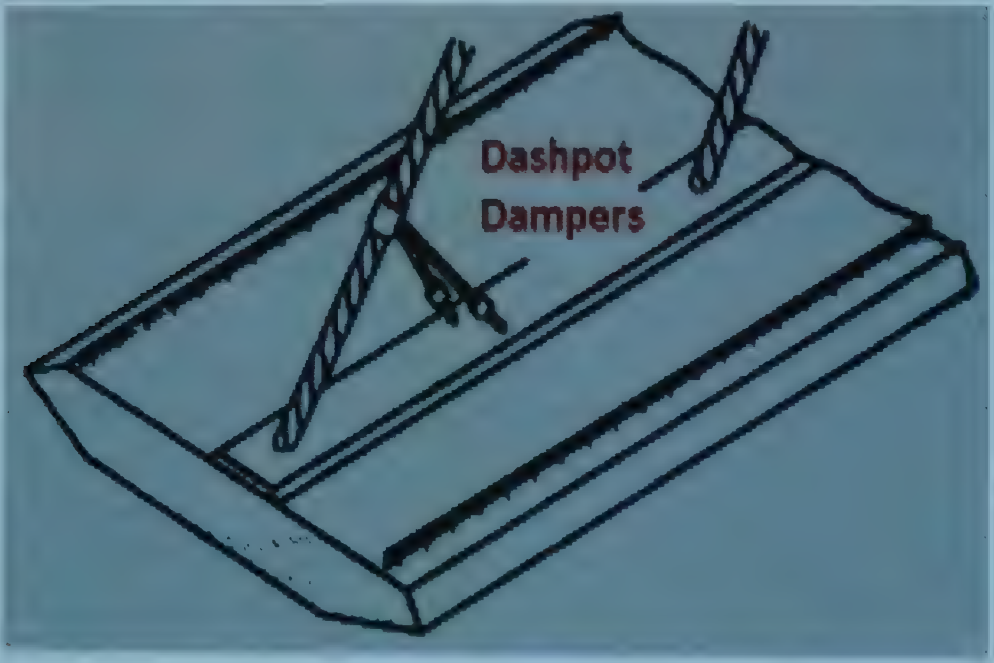
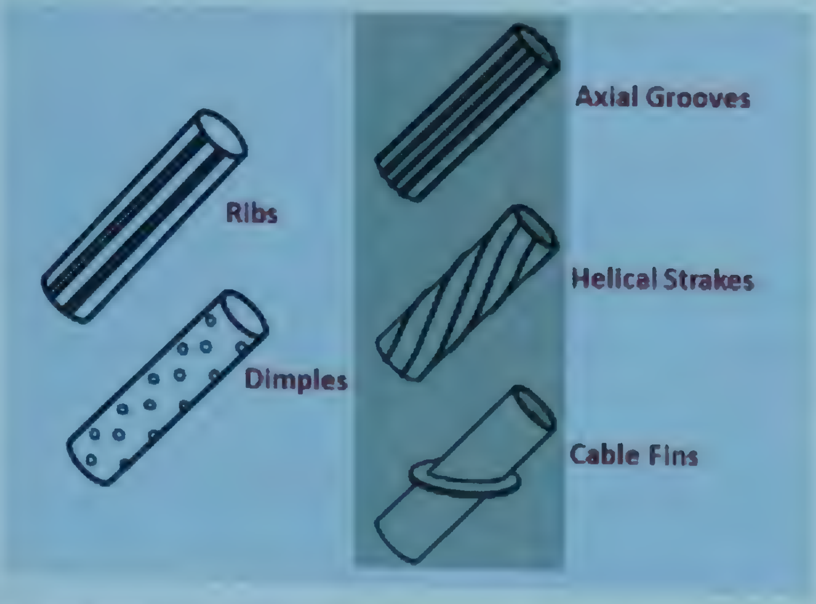
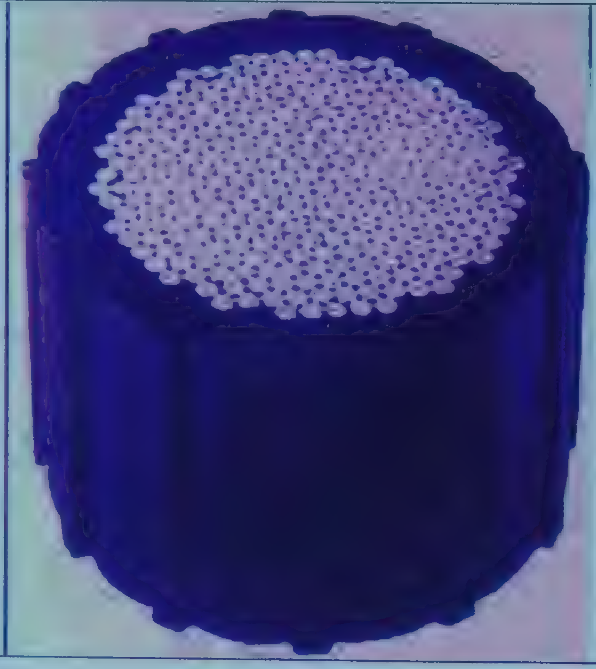


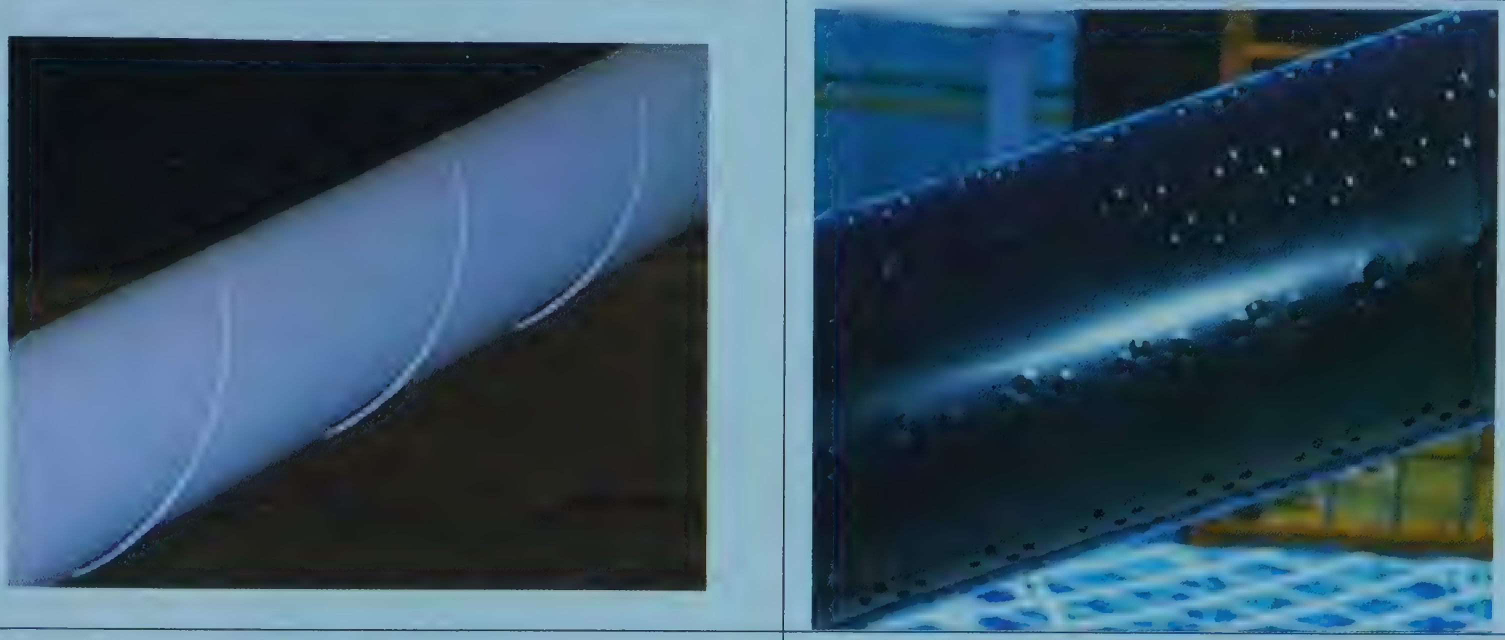
Fig. 10.5 Dampers installed on the stay



(a) Surface Treatment to Control Rain-Wind Vibration



(b) Axial Grooves on HDPE Tube



(c) Helical fillets on HDPE Pipe

(d) Indentations on HDPE Tube

Fig. 10.6 Stay Vibration control Measures

The preferred and proven treatments are either a system of double helical fillets running along the cable or a system of indentations or dimples in the surface. Protruding strips can also be provided. The double helical fillet treatment is usually employed for PSS cables while the dimples are used for NPWS cables. This different approach is due to suppliers' preference and the different manufacturing techniques. The purpose of the treatment is to prevent the formation of continuous rain rivulets running down the cables. These measures have proved to be effective in minimising the risk of large amplitude rain/wind induced vibrations. The actual shape and dimensions of the fillets and indentations must be optimised to minimise the effect on the drag of the cable. For both systems, wind tunnel tests have shown that with optimised dimensions only a very small increase in drag coefficient occurs.

Fig. 10.5 & 10.6 depicts the countermeasures for the type of vibrations described above in **Sections 10.4.2 & 10.4.3.** Besides the above measures, cross cables are employed, if it is required to change the vertical mode frequencies of the cables. Cross cables are often regarded as a last resort in suppressing vibrations as they, apart from requiring maintenance themselves, complicate inspection and maintenance of the main stay cables. However, it is recommended that the stay cables should be prepared for possible future installation of cross cables. For structures using LCS and NPWS cables no preparation is generally required for the cables due to the fact that they are compact and will be able to accept the required clamps for the cross cables. For PSS cables special collars are required at interfaces with the cross cables. **Fig. 10.7** is an example of such an application.

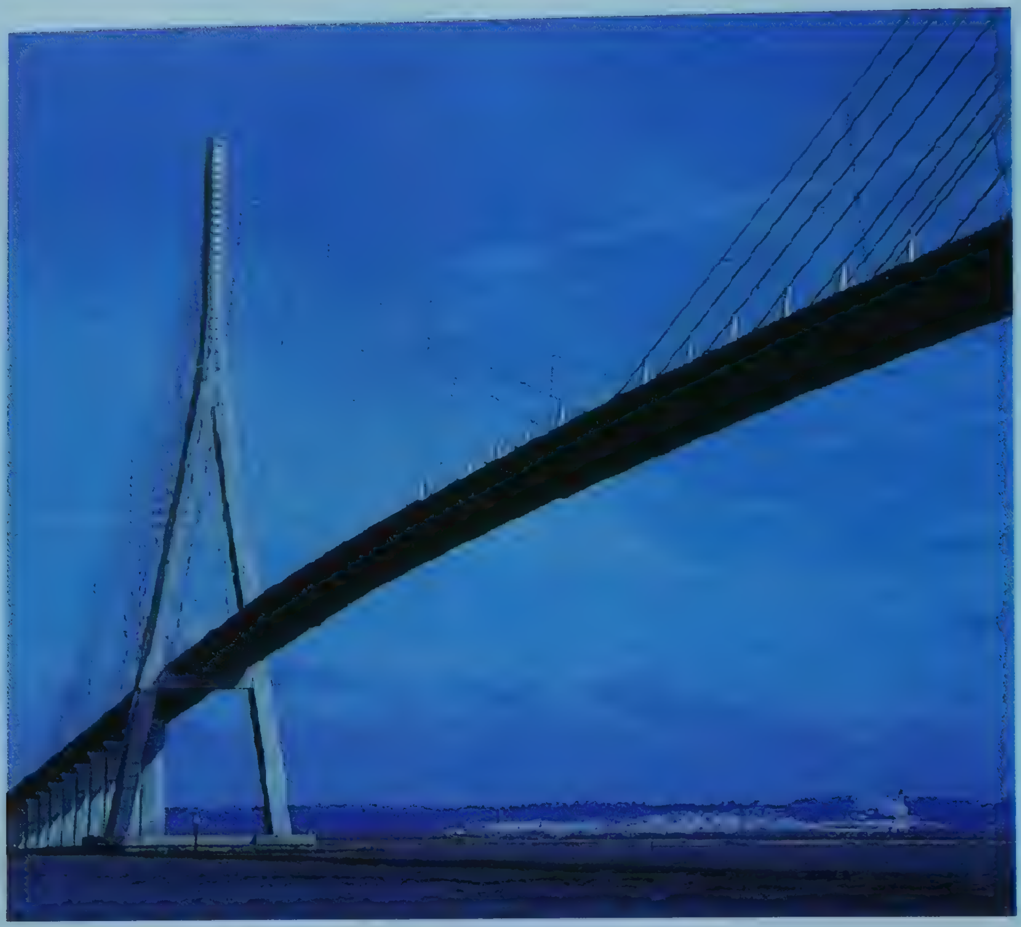


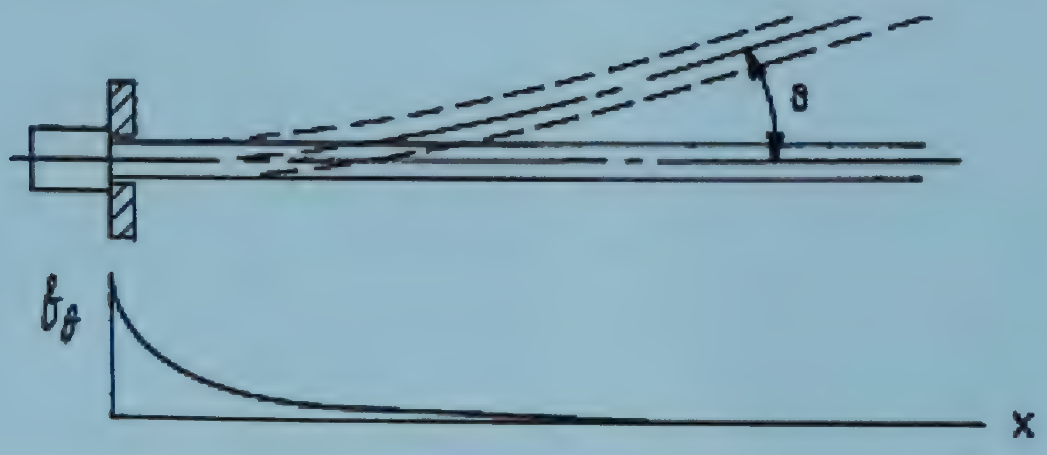
Fig. 10.7 Cross Cables (Normandy Bridge)

10.5 Bearing Plate, Recess Pipe Installation

The stay cable recess pipe is installed when casting the segment. Normally, the recess pipe is bolted or welded to the anchoring bearing plate at a right angle.

Great accuracy is required when placing the guide pipe as it will dictate the stay cable alignment.

The vertical and horizontal angles are especially critical, as offsets would create bending stresses (**Fig 10.8)** in the stay cable. The anchorage adjustable nuts or shims are in contact with the bearing plate. The bearing area must be perfectly flat to avoid stress concentrations and deformation of the anchorage socket or adjustable nut.



$$f = 2\alpha \frac{d}{4\lambda}\sqrt{\frac{EF}{A}} \times e^{-KX} \qquad K = \sqrt{\frac{F}{EI}}$$

Fig. 10.8 Stay Bending

10.6 Stay Cable Pipe Installation

The main difference in installing the various cable types is whether the cables are erected as fully prefabricated units (LCS and NPWS) or are site-assembled units erected strand by strand (PSS).

Erection of fully prefabricated cables will require transport and lifting of heavier units and deployment of heavy lifting equipment.

Erection of PSS cables requires only light equipment, but is considered to be slightly more time consuming.

As the cable installation is not normally on the critical path, the main difference from a costing point of view between the systems is the plant associated with the installation process, where the lighter equipment used for the PSS cables is a clear advantage.

Steel pipes have to be butt welded. Many of the welds can be done with the pipe lying flat on the deck, while others have to be done with the pipe in an erect position. In either case, the detailed butt weld procedure must be developed and approved. Random x-ray check of the welds is recommended.

Polyethylene (HDPE) pipes are welded together by fusion, using a special device to maintain proper pipe alignments.

Different systems are used to temporarily support pipes prior to strand installation and stay-cable stressing. These have to be designed to prevent excessive bending of the pipes. The stay cable pipes can be supported from the previously erected pipes, with messenger cables or highline, or by a few stay cable strands inside the pipe.

10.7 Installation & Stressing of Stay Cable Components

Stay Cable supplier instructions should preferably be followed strictly for the installation of the other stay cable system components. Guidelines for the same are enumerated in **Section 5.5.**

10.7.1 *Stressing of Strands*

For stressing of Multistrand Stay Cables, two different methods are available:-

a) Single stressing system using monojacks (Isotension method)

b) Multistrand jack stressing (gradient jack).

10.7.1.1 Single Stressing System Using Monostrand Jack (Isotension Method)

Parallel strands stay cables are installed normally strand by strand. Due to flexibility of deck and tower, the force of the individual strand of a stay cable varies during the stressing operation. To ensure uniformity of force in all strands of a stay at the end of stressing operation, a method has been developed by Freyssinet, called Isotension. The principal of the Isotension method may be described as follows:-

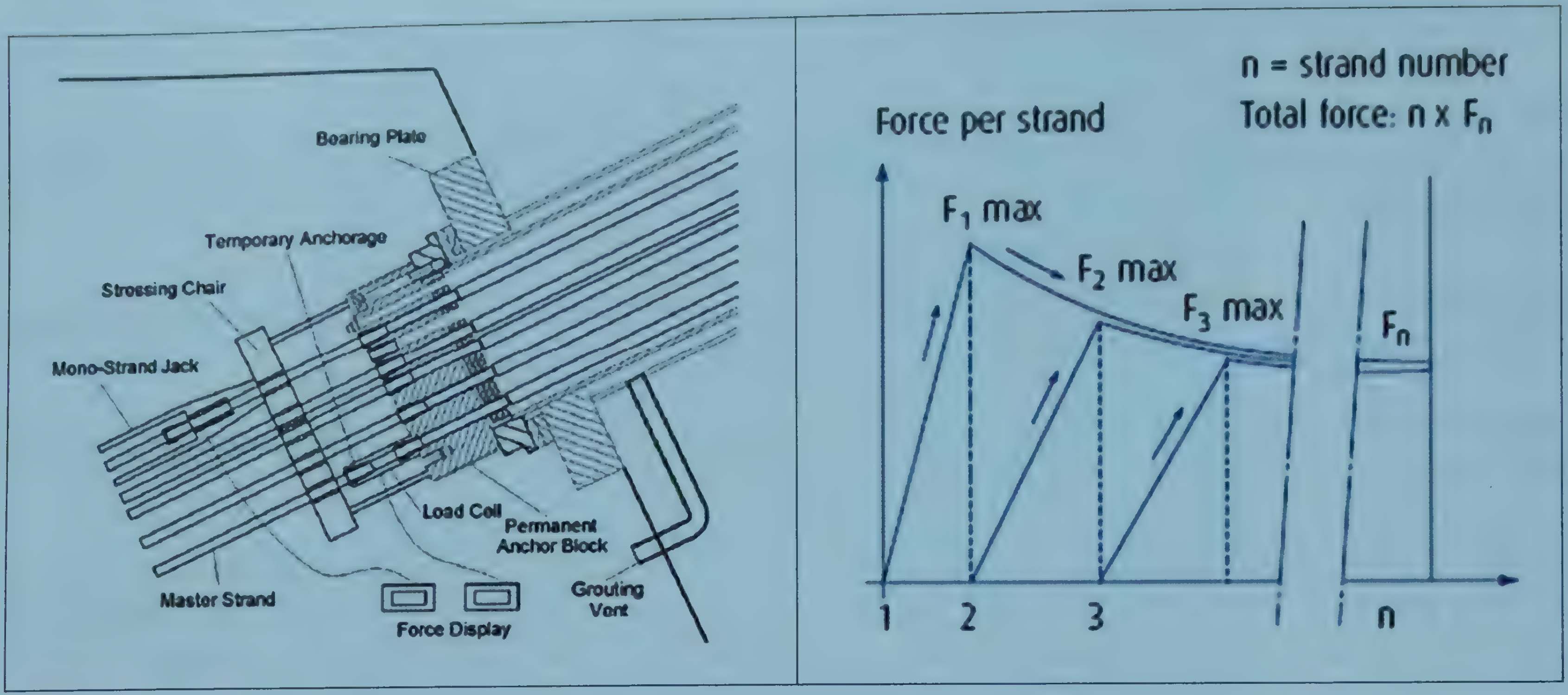

Bearing Plate
Temporary Anchorage
Stressing Chair
Mono-Strand Jack
Load Cell
Permanent Anchor Block
Master Strand
Force Display
Grouting Vent
n = strand number
Force per strand
Total force: n x F_n
F_1 max
F_2 max
F_3 max
F_n
1
2
3
i
i
n

Fig. 10.9 Isotension Method and principle diagram

- The first strand ("the master strand") is fixed to one of the stay cable anchorages and its other end is threaded into the other anchorages and then cut and stressed to a pre-calculated force which has to be higher than the desired target strand force (when all strands of the stay cables are stressed). Such pre-calculated force needs to be arrived at based on analytical model taking account of flexibility of superstructure and tower.

- The wedge of the master strand is not yet engaged to the stay cable anchor block. The master wedge is instead wedged to a special single anchoring device provided with a load cell which gives the reading of the current master strand force.

- The second strand is then installed in similar manner, cut and stressed. The stressing is done as for the master strand, with a mono-strand jack equipped with a load cell identical to the one indicating the force of the master strand. As the second strand is stressed, the force in first strand decreases and the stressing operation is stopped when the readings of the two cell are identical. The second strand is then anchored/ wedged to the permanent anchor block. The two strands are equally stressed.

- The third strand is then installed and stressed until its force reaches the force of the master strand (which decreases jointly with the force of the second strand). The three are identically stressed.

- The same operation is repeated until the last strand of the stay cable is installed. The last reading of the stay is then to be compared with the target theoretical force (**Fig. 10.9**).

10.7.1.2 Multistrand Jack Stressing (Gradient Jack)

Final stay cable force may then be adjusted by the multi-strand jack, if necessary. Elongation of stay cable which can happen using multi-strand jack is very limited in the stressing end anchorages design and its requirement must be defined by designer in the beginning of job to the anchorage supplier.

10.8 Cable Replacement

Replacement of the LCS and NPWS cables has to be done cable by cable and substantial traffic restrictions will be required for a considerable length of time. Handling of the cables will require the use of heavy lifting gear, both at deck and tower level.

Replacement of PSS cables can be done strand-by-strand using basically hand held equipment requiring only limited or no traffic restrictions.

PSS cables have a clear advantage when comparing replacement procedures and as it is usual to assume a 50 to 60 years design life for the cable system. At least one total replacement of the cable system during the life of the bridge must be assumed.

10.9 Protection from Fire and Vandalism.

Vehicle fires can happen occasionally on bridges. Since bridges are in open area and well ventilated, hence they are relatively less exposed to high temperature rise in the event of fire of a truck/cars. However, if the truck carries hydrocarbons and catches fire, it could cause a significant rise in temperature of a bridge in the event of fire. If the truck burns near a cable stay, structural stability of structure must be ensured by considering the failure of one cable stay.

However, the case may be different for an cable stay bridge with set of closely grouped stays or large number of trucks carrying hydrocarbon (near oil refineries) and covers a large numbers of stay cables. In such special condition, provisions need to be done so that temperature rise in the main tensile elements of the cable / anchorages must be retarded for a period, it would take to extinguish the fire.

Study of vehicle's base fire hazard may be required to carry out to asses required fire rating of cable stay system. Such study include:-

- Fire characteristics of vehicles intended to be passed over bridge
- Heat transfer to the structural elements via radiation heat flux
- Temperature increase of the structural elements
- Resulting material and mechanical response to the elements
- Range of fire type, size , location to develop envelope of performance
- Effectiveness of fire protection measures
- Fire Ventilation

Special insulating material have been developed by stay cable supplier's for such purpose which enhances fire resistance to main tensile element (MTE) by protecting MTE to a level so that it can carry applied design load. Fire resistance qualification tests shall be required to be carried out to establish fire rating of cable stay system **[3]**. The 30-minute fire rating for fire resistance stay cable including anchorages is a minimum standard for fire resistance. However, if the owner feels fire rating demand for a project specific is more than 30-minute, the stay cable system supplier needs to demonstrate with a longer duration of fire rating .

It may be noted that in case of fire event, the entire cable system needs to be replaced irrespective of usage of such insulating material of required fire rating.

The design of transition pipes must allow of a special steel pipes protecting the stay cable against vandalism also. The details shall be designed such that these steel pipes do not impose force or deformations to the stay cable which could be harmful for the stay cable performance.

10.10 Anchorages for Cables

The two ends of cable stay are called as anchorage zones. These consist of:

- Anchorage which transmits the tension of the cable stay to the attachment parts of the structure (deck, cross beams gussets, pylons etc..)
- Transition zone extends from the anchorage to the start of the free length of the cable stay. The transition zone comprises of deviators, transverse guide systems, external dampers and sealing system.

  The anchorage zone and the transition zone generally fulfil the following functions:-

- Transfer of Cable Stay Force- Anchorage consists of mechanical part (wedges) designed to secure the main tensile elements of the cable and transmit their forces to the attaching parts of structure.
- Filtering Out Angular Deviation- Anchorage may comprise cable guide systems in order to partially or totally prevent angular deviations of the cables extending to the anchorage head. Hence appropriate system be provided to limit or eliminate the effect of cable stay deviations at the anchorages head i.e to filter out changes in angle between the cable end and anchorages.
- Possibility of Adjustment- Anchorage has provisions for moving the anchorage head with respect to structure to provide cable stay tension adjustment and initial cable stay directional adjustment.
- Corrosion Protection and Water Tightness- The design of anchorage must allow the two barrier corrosion system of strands to be extended up to anchorage plate without any break. Certain layer of protection specified for free length can not be fully continuous from end to end of cable. This applies in the transition zones wherein sheathing of prestressing steel is removed inside the anchorage. Whenever one of the layers of protection is interrupted it must be replaced with an equivalent layer of protection.

- Provisions shall be made in anchorage to drain out water to clear out ingress through any stay barrier or stay-anchorage connection.

- Cable Replacement- The design of anchorage allows the replacement of complete stay cable as a whole (for LCS & NPWS cables) or in parts strand by strand (for PSS cables).

10.11 Anchoring / Passage of Stay Cables in Pylon Body

Two dispositions exist for the transition through the pylon head. The first cuts the continuity of the stay cable. In this case, there are 3 typical cases which are defined in (**Fig 10.10)** as cases A, B, C. The second keeps continuity of stay cable within the saddle, shown as case D .

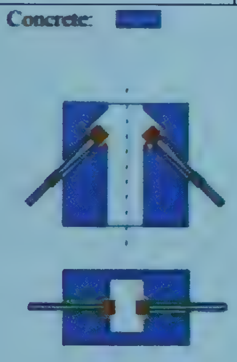
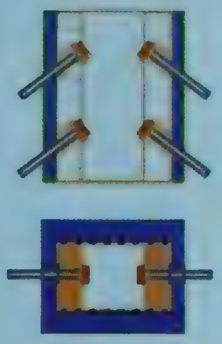
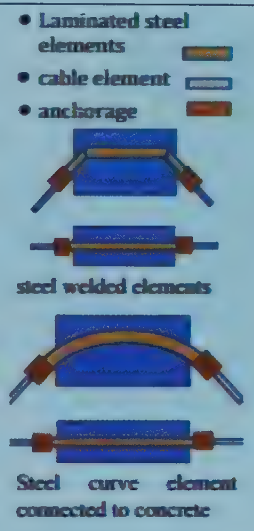
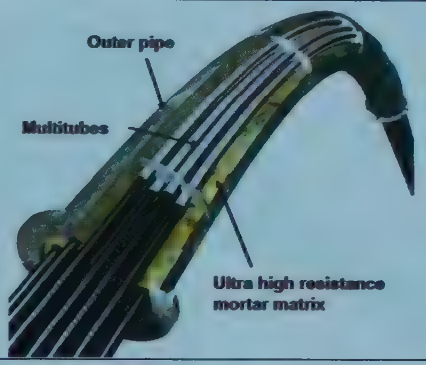
Type	A	B	C	D
Elevation Plan			 • Laminated steel elements • cable element • anchorage steel welded elements Steel curve element connected to concrete	 Outer pipe Multitubes Ultra high resistance mortar matrix Courtesy-M/S Freyssinet International Courtesy-M/s VSL
Comments	Cables anchored inside concrete pylon	Cables anchored inside composite element or in Mast head steel	Cables anchored in cast-in steel element	Individual holes for individual strands of cables passing over saddle in concrete pylon

Fig. 10.10 Anchorage Configuration in Pylon

The A and B type of steel anchorage box structure at the head of pylon allows an easy maintenance, and as such these are extensively adopted in cable stay bridges. The working space which is necessary for installation of anchorages inside of pylon body requires internal shaft which increases pylon size. In order to take all equipment's up for stressing of stay cables and further inspection and maintenance at such anchorage location also requires access staircase, platforms etc through shaft inside pylon body.

Another option is where the stay cable is not continuous and is anchored on each side, on an external nose section where the cable can be anchored by means of standard anchorage. The C type of anchorages of stay cables can be done outside of pylon body. In such case, multiple numbers of curved profiled built up welded steel elements (with external nose element at its both end on which the stay cable are anchored) are pre-installed in the pylon body for all pairs of stay cables which are required to be anchored on either side of pylon. There is a internal steel link saddle for every pair of cable anchored on either side of pylon. For such case, it is always preferred that stressing of pair of cable stays (on either side of pylon) anchored at both end of internal steel link saddle shall be done simultaneously from its lower end (anchored to deck). This will induce direct tension in internal steel link saddle, thus creates an inward pressure to concrete body of pylon head. Any differential force in pair of stay cable anchored at both end internal steel link saddle get transferred to pylon concrete body through welded shear connector provided on outside surface of internal steel link saddle.

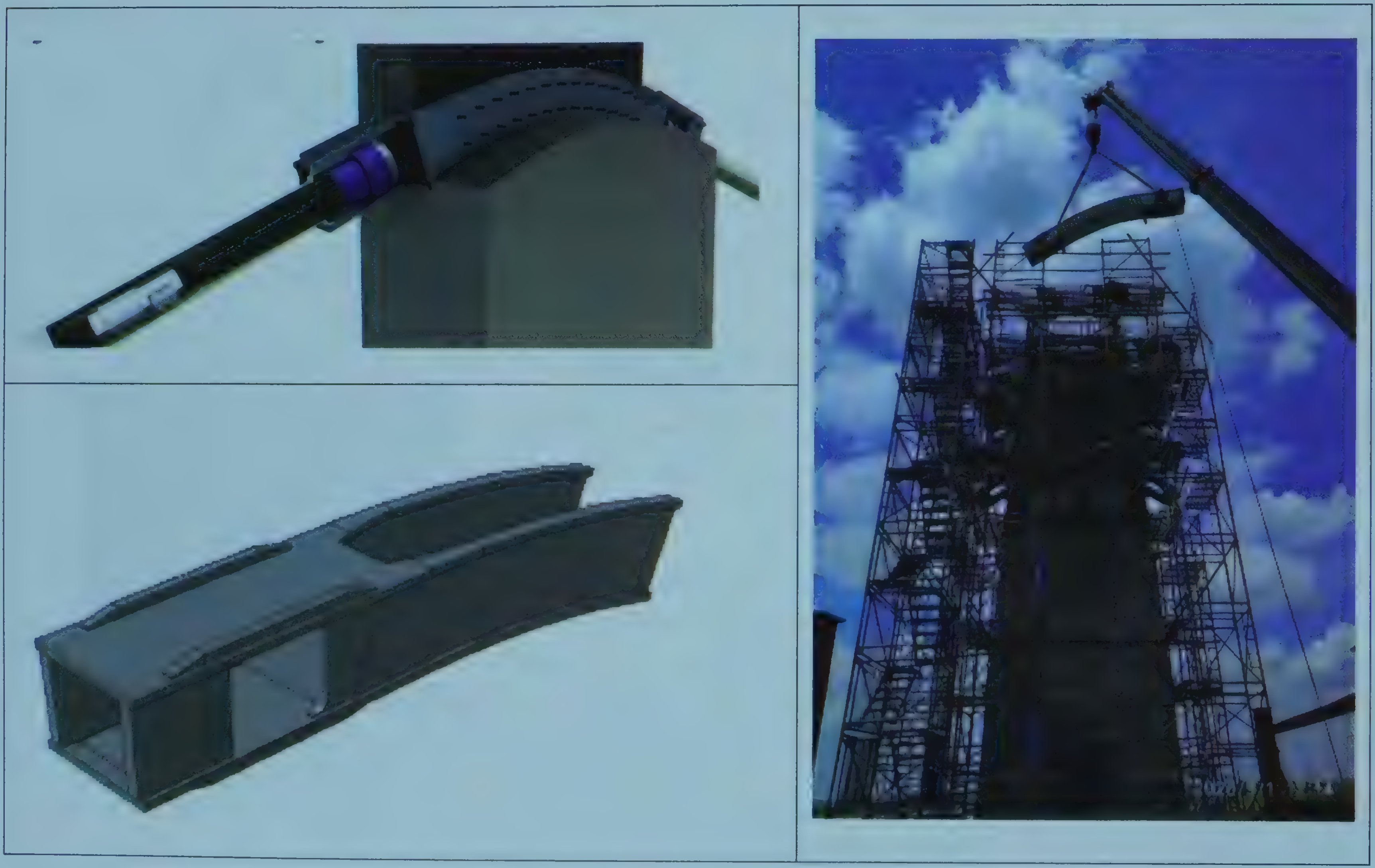


Fig. 10.11 Internal steel link type saddle

Lateral openings can be outfitted to allow an easy exchange of single strands or even whole cable if required. **Fig. 10.11**. In concrete pylon, such saddles have added structural advantages as: -

- Shear studs at the steel beams inside the pylon easily transfer differential loads within the concrete although a proper check of elongation vs shear stud loadings needs to be performed.

- Friction, bond stresses and mechanical wear due to strand deviation are avoided.
- An unbalanced installation sequence as well as the use of two different cable sizes at one anchor box becomes possible.
- The left and right side cables of one box can be differently inclined, and there are no limits in the respective deviation radii.
- The design of the anchor box is fully in accordance with conventional steel construction standards with element always in tension.
- If any inspection and maintenance of the corrosion protection at the cable anchorage is required, it can be performed easily from outside the pylon structure.
- Possibility to design a slender and elegant pylon
- No necessity of an access inside the pylon

In some cases, instead of anchoring the cable in pylon, cable stays are allowed to continuously pass thorough pylon by providing a saddle (D type in **Fig. 10.9**). There is a saddle per cable. These allow the continuity of tensile elements without gap and limit the sizes of the pylons, and, provide the anchorage for the cable while avoiding their slippage. The advantages provided by saddles as reported generally are:

- Saving of anchorages
- No tension elements buried in concrete
- No necessity of an access inside the pylon
- A possibility to design a slender and elegant pylon

However, saddle seldomly used in cable stayed bridges as inclination of stay cable with horizontal is too large and difficult to negotiate. However, if adopted, it should be preferred option for bridges in non-seismic zones. Saddles, which are solely depend on friction to block differential forces in stay cables on either side of saddle shall not be used in high seismic zones. If adopted, detailed structural investigation of the bridge shall be carried out using site specific response spectrum or time history analysis using spectrum compatible time histories.

Saddle is only an alternative option to replace the stay cable anchorages in pylon of relatively small and slender structures mainly for the reasons of space requirements. However, for large and significant structures, wherein space requirements are not in very critical in pylon structures, anchorages at the pylons are always preferred for their proven performance in terms of tensile and fatigue strengths. Anchorage also offers advantage in terms of ease of installation and possible future replacement which is very difficult in saddle system.

If at all, saddles (with individual hole for individual strand only) are proposed by designers. In that case following shall be ensured:

- Blocking of differential forces in stay cable on either side of saddle (pylon) by internal friction and transfer to pylon using mechanical anchorages. Slippage shall be avoided at any state of the structure including at ULS condition and saddle shall be tested for the same.
- Detailing of the pylon structure around the saddle using reinforcement, if needed according to the applicable design code to ensure the transfer and diffusion of the differential forces in the pylon body structure.
- Saddle system performance shall be at par with cable anchorages. The bending of the cable at the entry into the saddle under permanent and variable loads shall be adequately considered by providing guiding devices / bending filters.
- Continuity of corrosion protection of cables through the saddle by different protection system inside the saddle which is equivalent to the one provided in the free length. The transition of main to equivalent layer of protection has to be carefully detailed to ensure leak tightness.
- Replaceability of the cable stay
- Prevention of fretting corrosion in between main tensile element and saddle components
- Capacity to withstand fatigue loading due to pressure and friction of main tensile element against saddle components

Because of the curved profile of cables in saddle zone, flexural stresses in stays are often very high and needs to be combined with axial tensile stresses.

For Saddle system, requirement of initial testing and suitability testing as given in **Section 10.3.2 & 10.3.3** respectively shall be followed.

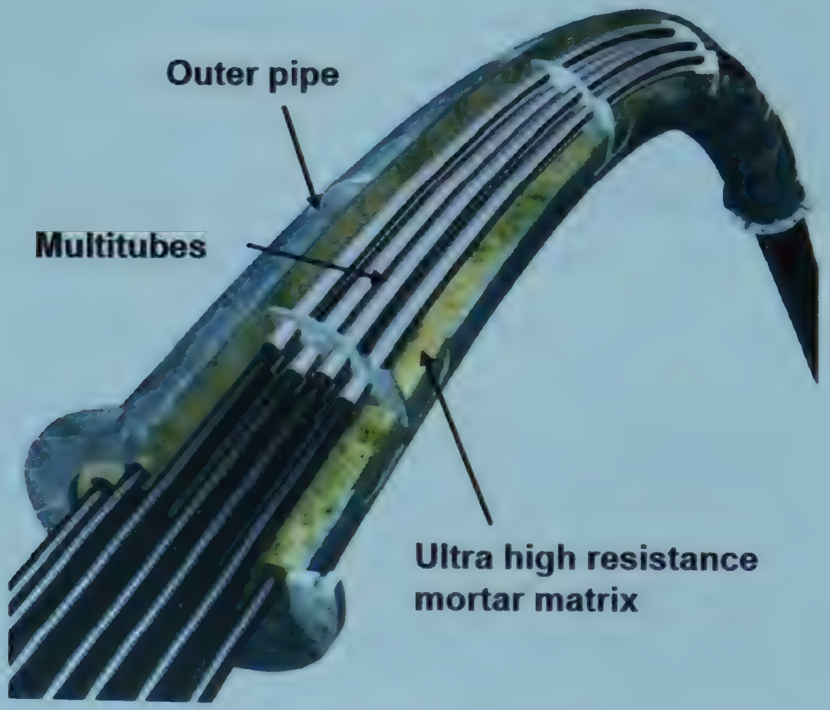


Fig. 10.12 Multi-Monotube Saddle

Conventionally, multi-mono tubes saddle **(Fig. 10.12)** are used with closely spaced diaphragm and space around tubes is filled with high strength mortar. Each strand of cable is allowed to pass through each curved profile tube. However, in such cases, strands are sheathed with specially designed resin or epoxy coated so that fretting corrosion of strand does not happen inside the tube.

Structural Saddles (**Fig.10.11)** in which main tensile elements are coupled at both ends shall be designed in accordance with standard applicable **[5]** and tested in accordance with conventional stay-anchorage system requirements.

10.12 Levels of Protection for Preventing Corrosion of Stays.

10.12.1 *Corrosion Protection of Stays*

The corrosion protection for the tensile steel element (the wires) must be provided with a certain redundancy in the form of two complementary nested barriers.

The first barrier generally employed in all systems is a hot dip galvanic coating applied directly to the wires and covering the full length of the cable without a break.

10.12.1.1 Parallel Strand System (PSS) Cable

For PSS cables each 7-wire strand is individually sheathed in an extruded HDPE. A corrosion-inhibiting compound, usually in the form of wax or grease, fills up the voids between the galvanised wires inside the sheath. The galvanisation and the extruded HDPE make up the two complementary barriers. In addition, the strand bundle is usually enclosed in an outer HDPE tube to give a smoother surface for the cable. Although this outer pipe constitutes an extra layer of protection it cannot be considered a complementary barrier as water can travel freely along the cable in case the tube is damaged.

Prior to the introduction of the individually sheathed strands a commonly used method was injection into the void between the galvanised strands and the outer HDPE tube with a corrosion-inhibiting compound, usually in the form of cement grout. However, a number of stay failures with this type of protection have been observed over the years and this is no longer a recommended method.

10.12.1.2 New Parallel Wire System (NPWS) Cable

For NPWS cables the tightly packed galvanised wires are surrounded by an extruded HDPE sheathing. The wire bundle is wrapped in filament tape prior to sheathing of the HDPE.

In order to achieve two complementary layers of protection the NPWS cables relies on the filament tape to prevent water to penetrate along the length of the cable in case of damage to the outer HDPE layer. It is therefore debatable whether the NPWS system fulfils the two level of Corrosion requirement unless an active corrosion-inhibitor fills out the voids between the wires.

10.12.1.3 Locked Coil System (LCS) Cable

For LCS cables the second barrier is either in the form of a paint system applied to the complete assembly or of an extruded polyethylene sheathing around the wire bundle. While an extruded sheathing might provide a slightly better protection than painting and might be preferred for aesthetic reasons, repair and inspection of a painted cable is easier and this is therefore the most used option. The interlocking Z-shaped outer wires form in combination with the self-compacting effect from the spiral arrangement a tight surface preventing moisture to penetrate and travel along the inside of the cable in case the outer barrier is damaged.

10.13 Dampers for Stay Cables.

Cable stays have quite a low intrinsic damping capacity. The most conventional way of limiting or eliminating cable stay vibration involves increasing their structural damping capacity by fitting special damping devices. The objective of a damper is to reduce the amplitude of a cable-stay vibration. However, the free longitudinal movement and free rotation of the stay cable at damper location is allowed for temperature elongation and force variations of the stay without any constraints.

The risk of vibration increases with cable stay length. Short cable stays (generally less than 80 m) generally involve no risk, other than parametric resonance in the case of a particularly unstable structure. Hence, such damping devices are generally needed, if the cable length is more than 80 m.

10.13.1 *Internal Dampers* ***(Fig. 10.13)***

These are of ring shaped and installed inside guide tube which in turn fixed to formwork tube. It means, they are always installed near the anchorage at lower end of cable stay. Internal dampers use to dissipate the energy either by the distortion of a dissipating material or friction of two interfaces. Various suppliers have different form of internal dampers to achieve their objective.

In case of internal elastomeric damper, dissipating material can be specially formulated rubber.

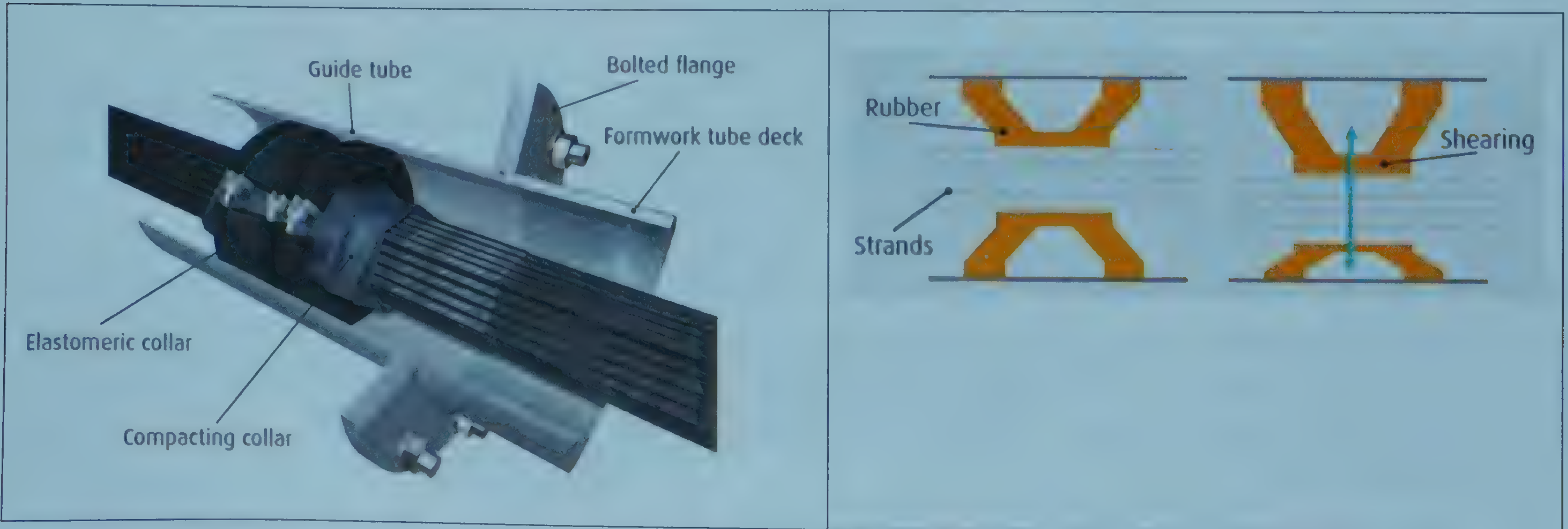


Fig. 10.13 Internal Elastomeric Dampers (Courtesy – Freyssinet Dampers)

In case of hydraulic dampers/viscous damper **(Fig 10.14)**, resistance is induced by the rapid passage of a viscous fluid through a narrow opening. The resistance can dissipate a large amount of energy leading to the damping of cable.

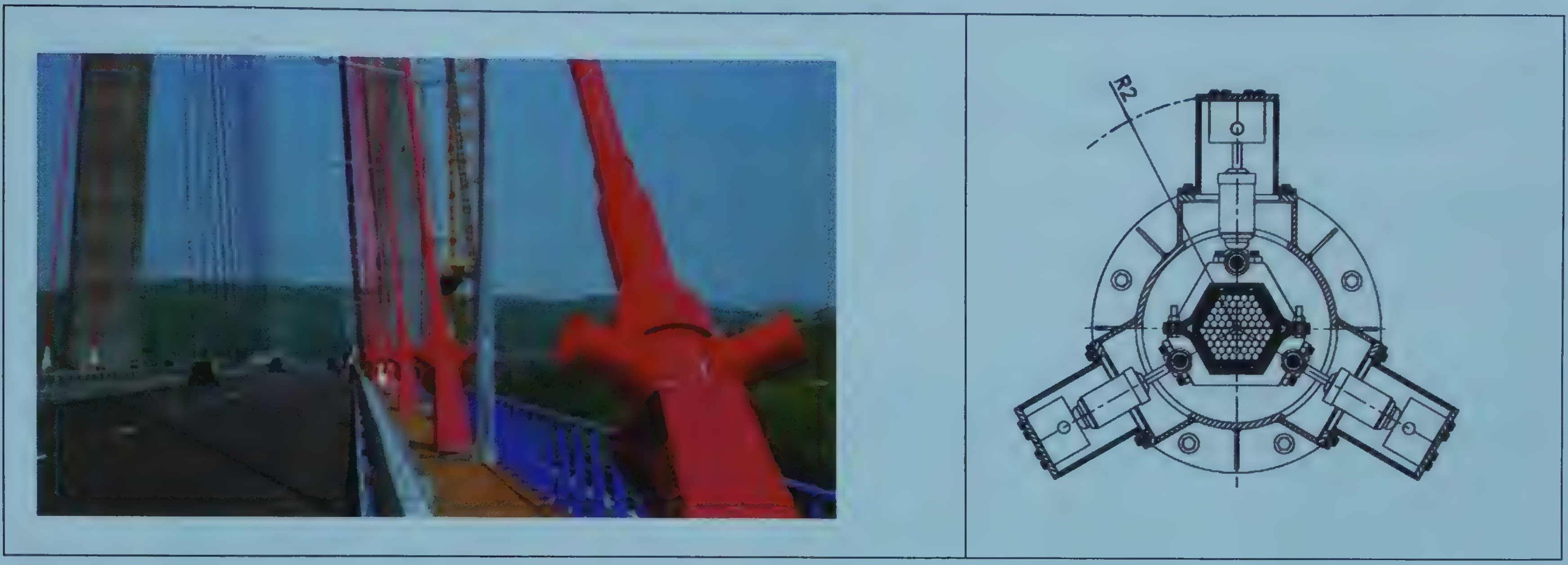
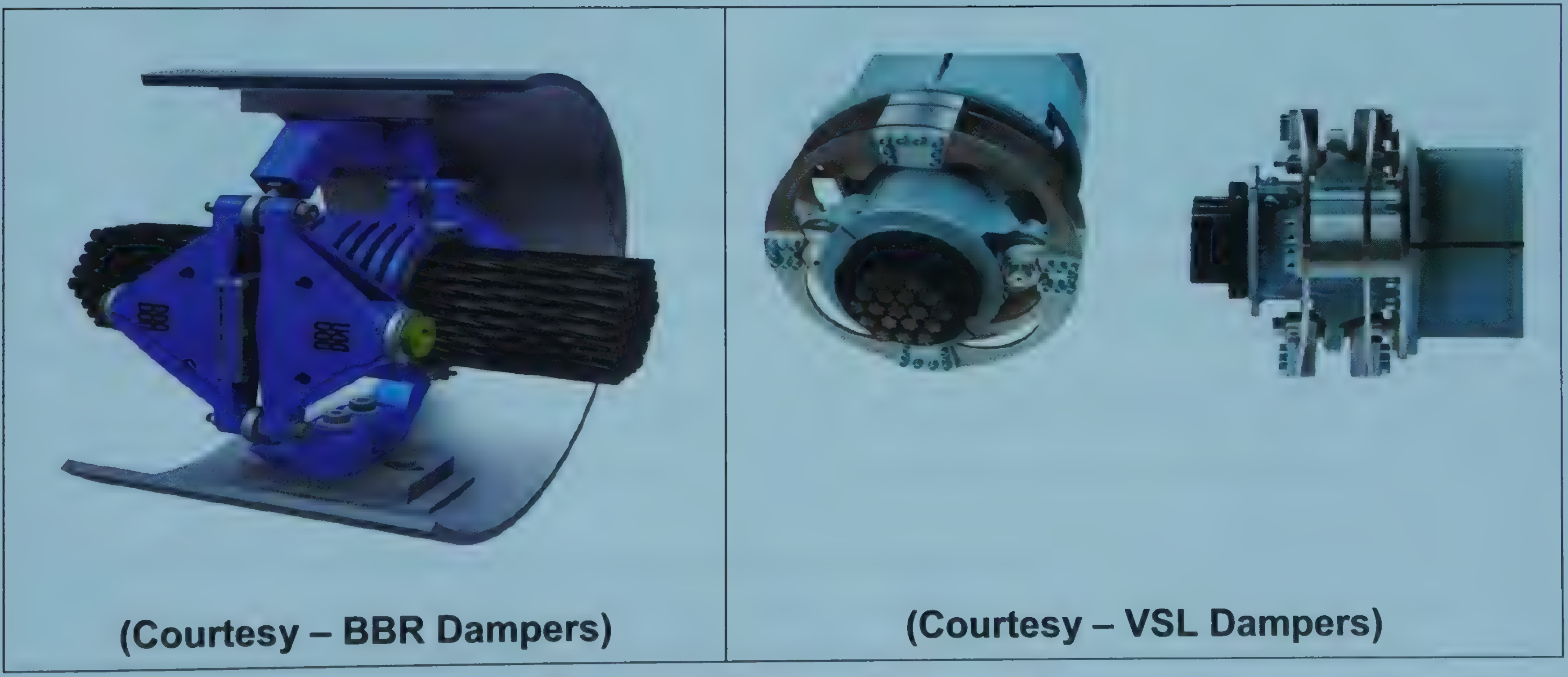


Fig. 10.14 Internal Radial (Viscous) Damper (Courtesy – Freyssinet Dampers)

In case of friction dampers **(Fig 10.15)**, sliding interfaces would dissipate the maximum of mechanical energy while keeping the threshold amplitude below a specified acceptable limit for the structure in terms of fatigue or user comfort.

Hence, friction Dampers is the efficient solutions for critical cases on long stay cables or where there is a risk of parametric excitation. Its non-linear damping characteristics are particular suitable to prevent cable vibrations independent of the vibration mode. The friction behaviour prevents the dampers from being activated at small amplitudes which are considered not to be critical for the cable performance. Once a certain amplitude is exceeded, the damper provides immediately an unrivalled amount of damping & preventing the build up of any large amplitude.



(Courtesy – BBR Dampers) **(Courtesy – VSL Dampers)**

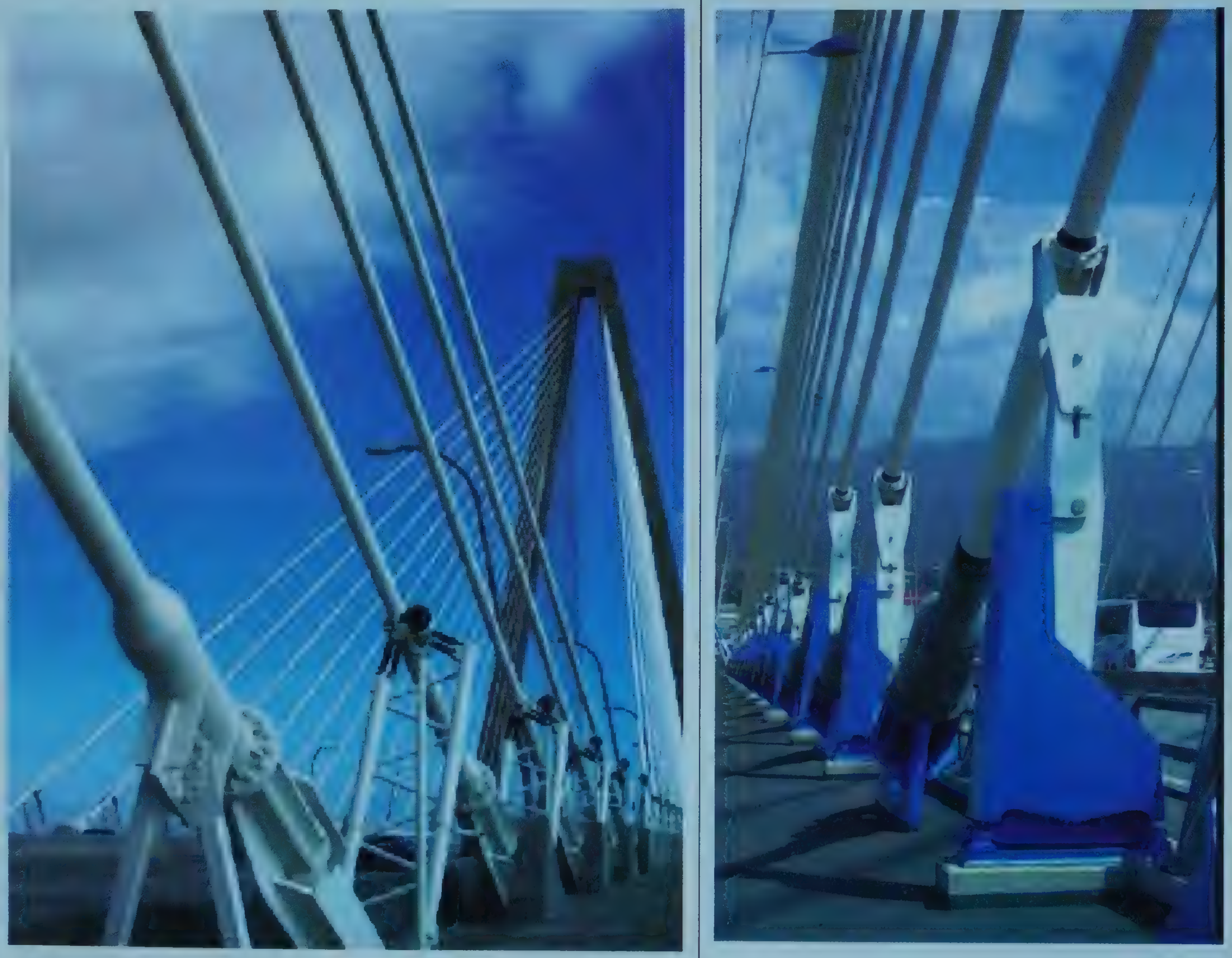
Fig. 10.15 Friction Dampers

10.13.2 *External Dampers* **(Fig. 10.16)**

For very long cables external dampers are more efficient than internal one. These are of two types:

Fixed External type: These are hydraulic devices exerting a transverse damping force on the cable near the anchorages. They are installed in pairs directly on the stay.

Pendular External type: It also uses a piston damper with a pendular lever system free to move around rod hinged on a fixed support.



Fixed type (with pair of piston)

Pendular Type

Fig. 10.16 External Damper (Courtesy – Freyssinet Dampers)

10.13.3 *Installation & Maintenance of Dampers*

Dampers are usually installed, once cables are structurally active and carry the permanent and superimposed loads of the structure. After installations, factors such as service load, traffic, wind and temperature modify the geometry of the entire structure and consequently induce relative rotations between the structure and the stay cables, which result in transversal and longitudinal movement at the damper locations. These movements are often larger than those imposed on the damper and the anchorages by possible cable vibrations. To ensure good damping performance, durability and safety, the damper, stay cables and anchorages must be analysed, designed and detailed as a whole. Consequently, both the stay cable and damping device should be provided by stay cable system supplier.

The damper and elasticity properties of a damper must be adjustable and adaptable to the each cable stay on a case to case basis.

Dampers undergo lot of wear and requires regular monitoring and, maintenance. Detailing of the damper shall be such that accessibility, adjustability & replaceability of either part of damper or as whole can be easily done.

10.14 Protection of Cable stays From Ice Accretion with De-Icing & Anti-Icing Systems

Cable supported bridges can suffer from ice and snow accretions that develop in specific weather conditions, predominately in cold climatic regions such as northern part of the country which are 2000 m above sea level like Ladakh, Lahul-Spiti, Shimla, Manali where ice generally covers the entire area including the cable/bridge structure.

High levels of humidity combined with frequent changes between passing cold and warm fronts results in high risk of atmospheric icing. These are formed from cloud droplets, rain drops, snow or water vapour. Duration of icing event is also important in development of ice accretion.

Pieces of accreted ice or snow fall from bridge members occurs

- when their weight increases beyond a value
- when temperature of outer surface of structural rises above 0°C , melting occurs at the surface-ice interface.

Block of Ice or Snow falling from cable stay / pylons may result in traffic accidents which can cause direct damages to passing vehicles and human safety at risk. Also this can result in fear / anxiety amongst drivers. Consequently, lack of successful de-icing or anti-icing measures may result in bridge closure which leads to traffic hindrance that can in turn lead to severe financial losses. Denmark's Great Belt Bridge was closed for 12 hours per year during falling of ice. Oresund Bridge, Severn Bridges of UK and Uddevalla Bridge of Spain have suffered similar experiences.

As a safeguard, different de-icing and anti-icing techniques **[4]** developed which are applied to bridge cables or pylons and it can be broadly classified into three categories:

- Mechanical methods based on breaking down of ice
- Thermal methods based on melting
- Passive methods based on natural forces

10.14.1 *Mechanical Method* ***[4]***

In this method, the ice is removed manually or automatically usually by closure of traffic.

Manual or mechanical method is carried out by physically breaking off the ice which requires workers to be lifted in gondola operated by crane along the cables. Such method can be only adopted, when there is no snow fall or string winds. Another way of manual method is by hitting hangers with base ball bat. This is being adopted in George Washington Bridge of New York.

There are many Automatic mechanical methods available. Electromagnetic-Impulsive De-icing (EIDI) systems employs by inducing strong and sudden magnetic force from a high current DC pulse through a coil, resulting in rapid acceleration and flexure to icing surface. This leads to debonding and breaking the ice. When such techniques are employed to cables by installing the coils along the exterior surface of the cables, physical outer diameter of cable is increased. This technique was used in Great Belt East Bridge.

10.14.2 *Thermal Method* ***[4]***

All thermal method works on basic principle of heating the ice-prone surface to prevent formation of ice or to generate de-icing. All thermal methods consume large amount of energy when operated.

Indirect heating by applying warm air to heat the ice prone area so as to melt the ice or prevent it from forming. Such method was applied in Uddevalla bridge, Sweden where a high pressure system pushed warm air through a small opening in cable protecting pipe and therefore kept the whole cable strand at temperature above freezing. Such method is successful but demands very high energy.

Others thermal solution is Pulse Electro-Thermal De-icing (PETD) method which uses short pulses of electricity applied directly to the ice-material interface by covering the surface with a thin electrically conductive film which is heated with milliseconds long pulse creating a thin water layer causing ice to drop off. This was used in one part of Uddevalla Bridge where both a cable stay and part of pylon was covered in PETD and very cost-effective as micrometer thin layer of ice is required to be melted.

10.14.3 *Passive Method* ***[4]***

Among many passive methods of anti-icing, none of the method completely hinders the formation of ice. However, such method can reduce the probability of formation of ice. Some of known methods are:

- Thermal absorbent coating which is effective only when sufficient radiation is available.
- Hydrophobic coating with surface tension and adhesion which is only effective in wet snow conditions.
- Solid icephobic coatings but still have ice-adhesion forces ten times greater than what is necessary for ice to detach under gravity or wind loading.

- Viscous products and anti-icing greases but have limited duration of protection as they loses their efficiency under the effect of precipitation.

10.14.4 *Conclusion*

To ensure traffic safety and to reduce long bridge closure, it is important to adopt a novel method against ice accretion and to mitigate the risk of falling ice and snow.

Passive system is not fully effective to mitigate ice accretion or snow.

Electromagnetic-Impulsive De-Icing (EIDI) system appear the most promising among mechanical techniques and worked very well when installed on Great Belt Bridge Hangers.

Although various thermal techniques are available but they consume lot of energy. However, Pulse Electo-Thermal De-icing (PETD) system shows good energy efficiency.

In India, nothing much has been done or in operation. It is high time to adopt EIDI or PETD system on cable supported bridges in snow clad areas.

10.15 Durability Design of Cable Stay Bridge

The durability of cable stayed structure can not be easily defined as each structure is unique of its own and factors governing the durability of cable stay bridge or cable stay system are:

- General design of structure
- Service condition
- Dimension of structure and particularly length / mass / tension force of its cable stays
- Environmental condition (Industrial, terrestrial or marine etc.)

Factors affecting durability must be analysed very thoroughly at the initial design stage or when cable-stay system is going through the qualification process.

For various categories of exposure class, ranges from inner room environment to very aggressive industrial atmosphere / coastal, offshore in all Indian conditions, multi-layer corrosion protection system as discussed earlier in this chapter satisfy, 100 years of design life of prestressing steel used in cables with high fatigue loading with present knowledge.

No scientific model is available, which can predict corrosion process over time as a function of exposure class or different level of fatigue. Hence, it is recommended to provide permanent multi-level corrosion protection to stay cable which is adequate for proven system of minimum 100 design life for all types of cable supported bridges **[2]**.

Based on ease of accessibility and condition of its replaceability of mild steel components of anchorages and saddle, cable stay system supplier shall present the corrosion protection approach of his cable stay system in accordance with **Table 10.1**.

Table.10.1 Corrosion protection philosophy for mild steel components of Anchorages & Saddle [2]

Aggressiveness of the environment (Very Severe Exposure Condition)				
Design Lifetime of the Cable Stay System	**Accessibility or replaceability of the components**	**Durability of the corrosion protection system**	**Design Life of initially applied corrosion protection system**	**Period between subsequent maintenance operations**
100 Years(1)	Replaceable	25 Years(1)	25 Years(2)	25 Years(2)
	Non replaceable Easy access	With maintenance > 100 years(1)	25 Years(2)	15 Years(2)
	Non-replaceable No access	100 years(1)	No Maintenance	

(1) Default value is 100 years (in case of higher value, it is to be specified by the designer/client)

(2) Minimum Value (actual value to be declared by the cable stay system supplier)

The cable stay system supplier shall define the type and thickness of coatings used on the mild steel cable components. He shall then demonstrate the durability of the mild steel components and declare in particular the required maintenance intervals and type of maintenance work anticipated such that the durability of the cable components is compatible with the design lifetime of the cables specified by the designer/client. He shall submit this information to the designer for approval.

CHAPTER - 11

BEARINGS, EXPANSION JOINTS AND SEISMIC PROTECTION DEVICES

11 Introduction

Cable-stayed bridges like suspension bridges are highly stressed structures coupled with fluctuating stresses due to dynamic forces (wind & seismic), thermal expansion / contraction, shrinkage & creep, etc. Such bridge structures undergo a large movement and rotation along-with high fluctuating load situations inducing demand for a structural support and protection system capable of catering to such forces and movements. By and far bearings and expansion joints for cable stayed bridges shall be designed as per the relevant IRC codes and guidelines. Some of the special devices and relevant precautions relevant to large span bridges are indicated in this chapter.

Given the length of continuous superstructure and the bearing arrangement, movement at bearing and expansion joint levels can be as large as up to 2 m. The bearing rotations are also often large, in the range of 0.03 – 0.15 radians. Hence, a highly durable & robust design & manufacturing of these devices is necessary.

11.1 Basic Concept of provision of Bearings and Expansion Joints in Cable Stayed Bridges.

This Chapter illustrates the utilization of bearings and expansion joints for long span cable stayed bridges with examples. **Fig. 11.1** shows a 350 m main span cable stayed bridge over river Chambal. Economical configuration of CSBs occurs when the main span exceeds 200 m which has been explained in **Chapter 4**.

Chapter 4 discusses the structural action of cable stayed bridges. It illustrates that in the service condition bearings are usually not provided to support vertical loads (DL, SIDL, LL) of the deck at the pylon location. However, since the side spans are usually about 50% of the main span, the end supports are subject to uplift which has to be countered by providing kentledge in some form. The space inside hollow box girder can used quite conveniently for the purpose of filling concrete, etc. Other possibilities are to increase the weight of part of the side span and also to use the weight of the approach span as a counterweight, as was done in the case of Naini Bridge **Fig. 11.2** as well as Tatara bridge, Japan **Fig. 11.3**. Another solution discussed in **Chapter 4** is to continue the deck beyond the main span into the shorter spans of the approach structure **Fig.11.1** (Chambal Bridge, Kota). In effect, it means that the end pier as well as other piers, when provided, must always be provided with bearings, sometimes to cater to uplift / tension. The bearings require capacity of large movements in the longitudinal direction even though the vertical loads may be small.

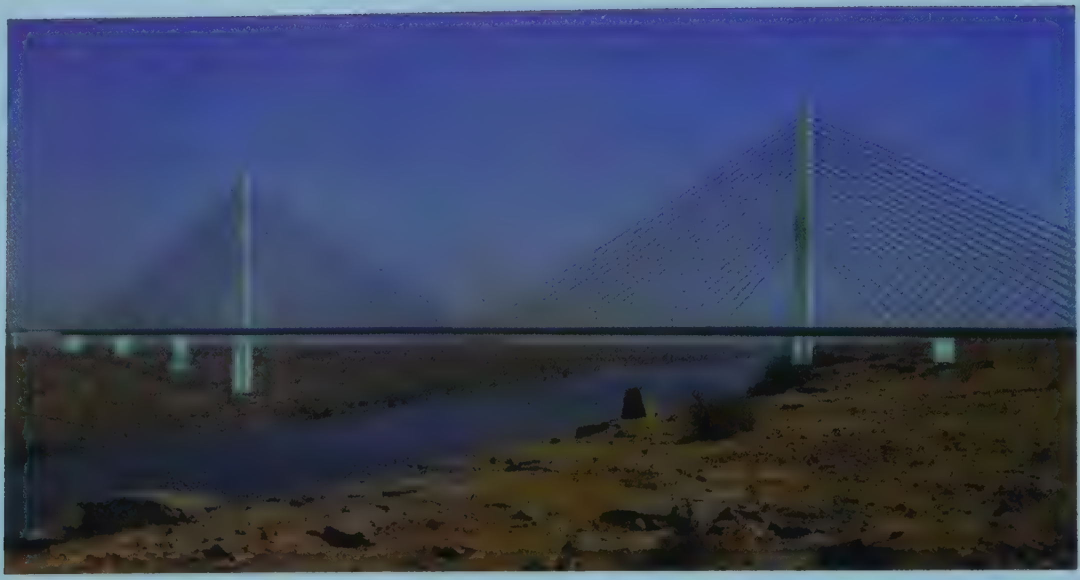


Fig. 11.1 Kota Bridge over Chambal, Rajasthan

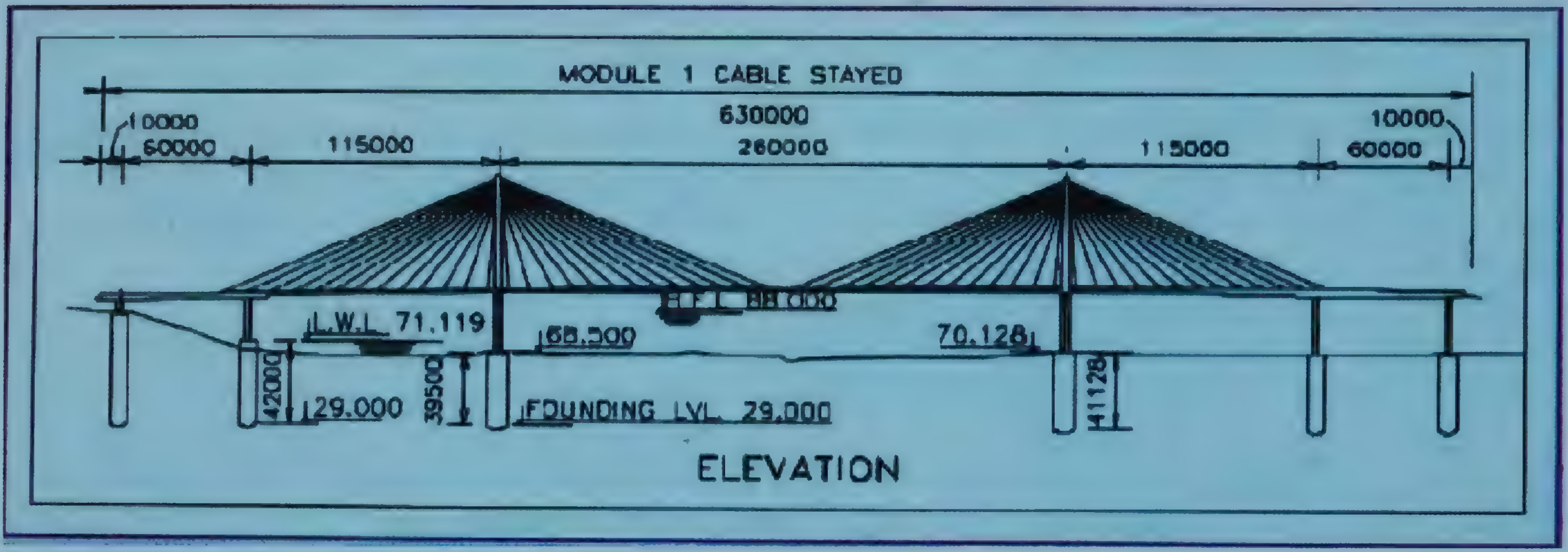

MODULE 1 CABLE STAYED
10000
60000
115000
630000
260000
115000
10000
60000
F.F.L. 88.000
L.W.L. 71.119
68.500
70.128
42000
29.000
39500
FOUNDING LVL. 29.000
41128
ELEVATION

Fig. 11.2 Naini Bridge

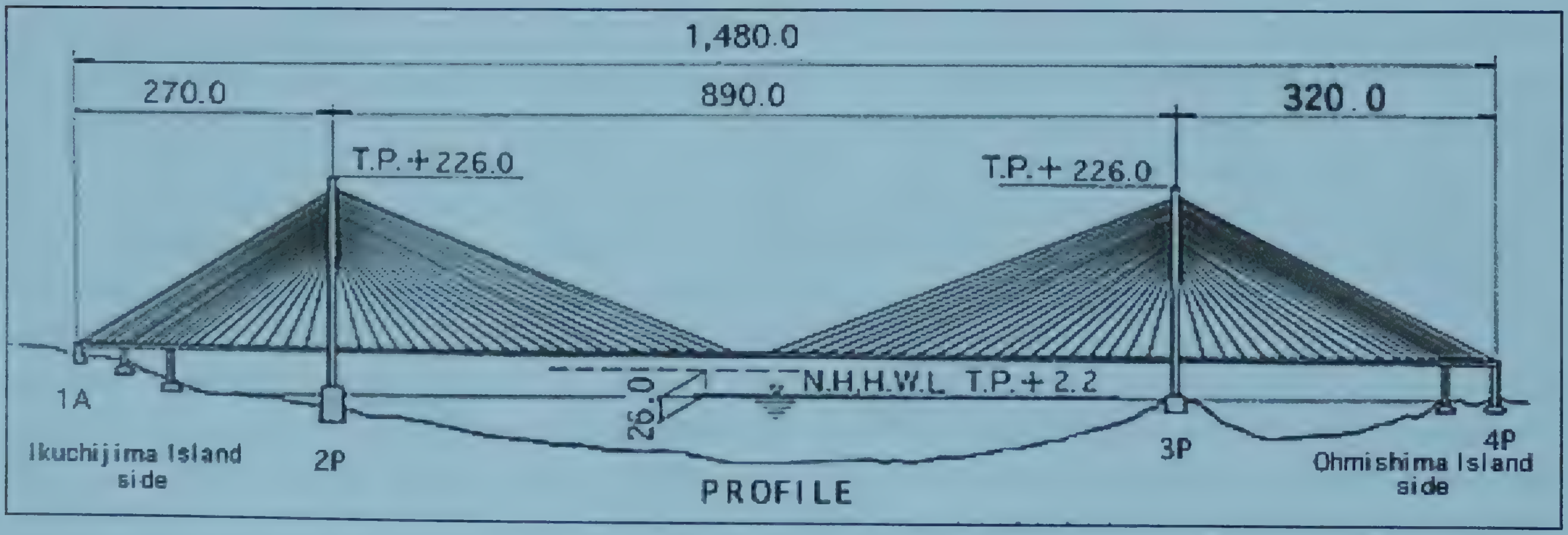

1,480.0
270.0
890.0
320.0
T.P.+226.0
T.P.+226.0
N.H.H.W.L. T.P.+2.2
26.0
1A
Ikuchijima Island side
2P
3P
4P
Ohmishima Island side
PROFILE

Fig. 11.3 Tatara Bridge, Japan

Since, the lateral forces due to wind and earthquake on the deck in the transverse direction of the bridge require to be taken care of before being transferred to the foundations, lateral restraints at the supports in **Fig. 11.4 & 11.5** (at all four or at least at two of them) in the form of suitably designed bearings in vertical plane are usually provided.

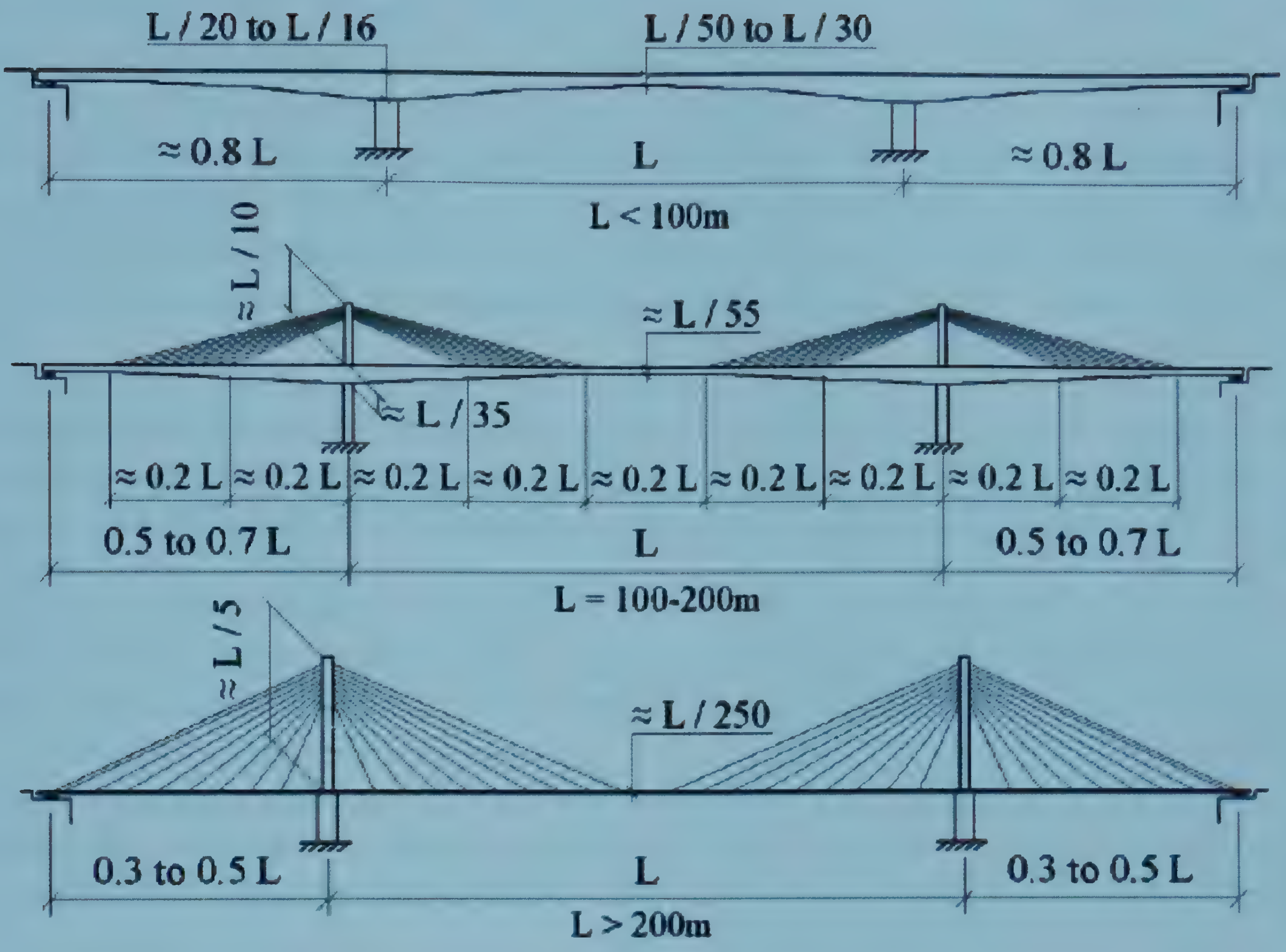

L / 20 to L / 16
L / 50 to L / 30
≈ 0.8 L
L
≈ 0.8 L
L < 100m
≈ L / 10
≈ L / 55
≈ L / 35
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
≈ 0.2 L
0.5 to 0.7 L
L
0.5 to 0.7 L
L = 100-200m
≈ L / 5
≈ L / 250
0.3 to 0.5 L
L
0.3 to 0.5 L
L > 200m

Fig. 11.4 Three Span Configuration

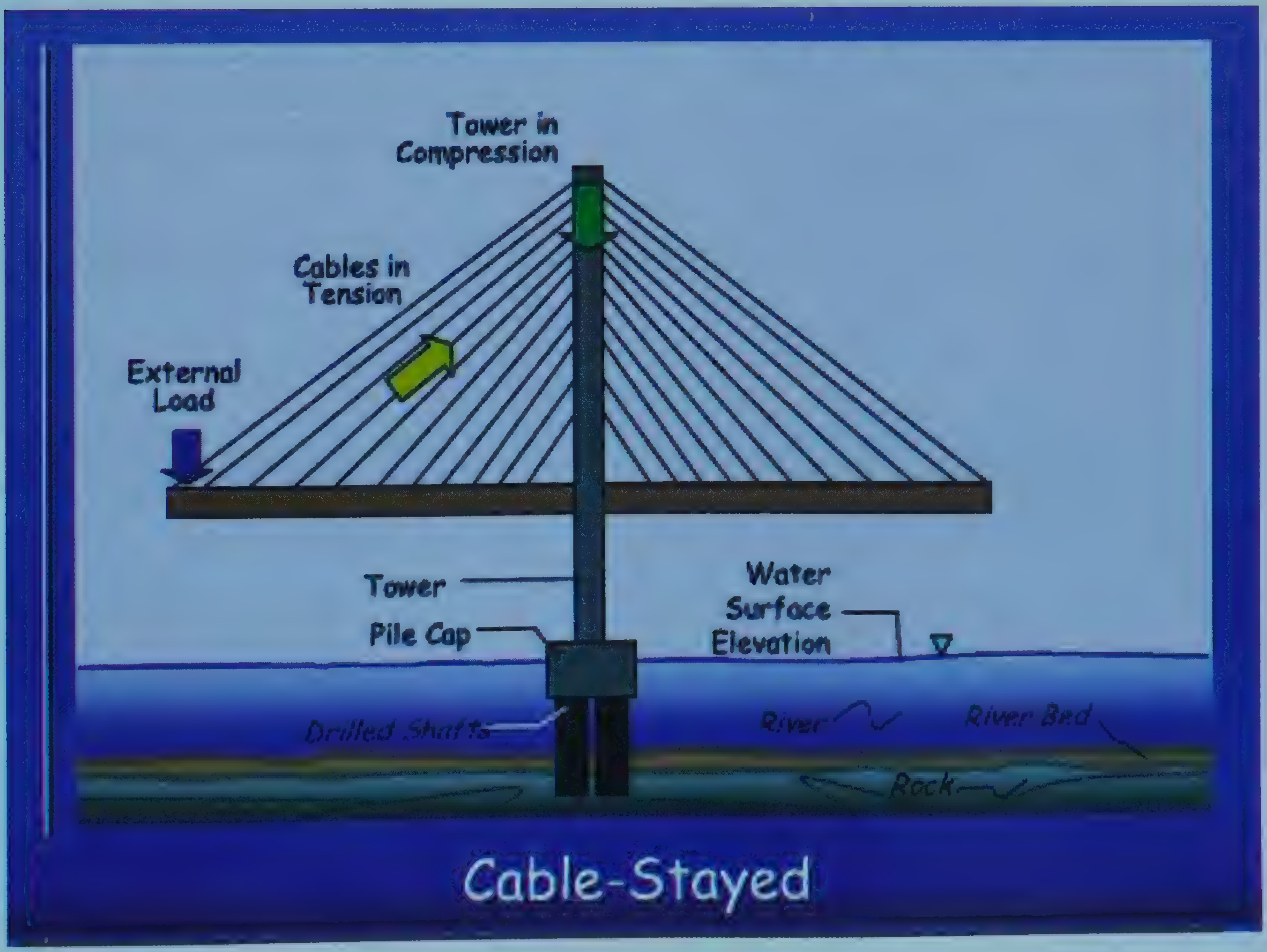

Tower in Compression
Cables in Tension
External Load
Tower
Pile Cap
Water Surface Elevation
Drilled Shafts
River
River Bed
Rock
Cable-Stayed

Fig. 11.5 Cable Stayed Bridge During Construction Stages

The Expansion Joints become necessary to enable the long continuous structure to breathe due to large thermal expansions & contractions, creep and shrinkage effects and also to cater to movements of the deck due to braking, earthquake and wind effects. Even for a small cable stayed bridge with main span of 200 m and side spans of say 100 m each, the distance between the expansion joints would be as large as 400 m. Movement from such large interspacing between Expansion Joint may be too large for a single strip seal or other regular types of expansion joints that are suitable for smaller movements and therefore Modular Strip Seal Expansion Joints or other suitable types become necessary. In high seismic areas certain special features in the expansion joints become necessary. These will be discussed later in this chapter.

Right selection of Expansion Joints, Bearings, Seismic Devices and vibration control devices becomes extremely important in case of Cable stayed Bridges where significant daily service movements, inducing fatigue and substantial accumulated sliding path occurs.

In this chapter the essential requirements specific to behavioural outcome of Cable stayed Bridges that translates to performance requirements of Expansion Joints, Bearings and Seismic Devices are discussed as below. Vibration control devices are discussed in other relevant chapters.

- **Wind effects:** Cable Vibrations as well as Vertical deck movements with fatigue effects. Increased stress / load levels, Accumulated sliding path in Bearings and Joints. Large wind forces into Bearings (lateral, up-lift, etc.)

- **Traffic effects:** Cable vibrations, Vertical & lateral deck movements and rotations with fatigue effects. Variation stress / load levels in the Bearings and Expansion Joints. Large accumulated sliding path in Bearings and Expansion Joints.

- **Seismic effects:** Anchoring forces to be transmitted through Bearings. Also, the very high movements.

In the following Sections, Bearings and Expansion Joints that are suitable for Cable Stayed Bridges are discussed.

11.2 Bearings

Bridge bearings must be able to transfer the loads & forces, hold the deck in position and allow expansion & contraction.

Caused by the dynamic actions and characteristics of Cable Stayed bridges, the Bearings system function in the bridge may require freedom in up to all 6 degrees. Cable Stayed Bridges impose repetitive rotation and translational movement at higher frequency, magnitude and velocity.

The selection of the appropriate bearing type shall be such as to fulfil the durability and service life demand of the Structure. This shall be fulfilled considering the accumulated movements, rotation cycles combined with wearing and fatigue effects. Bearing design shall cover SLS, ULS and simultaneous wind / seismic load cases whichever is governing without any compromises on their performance. Various types of bearings applicable to cable stayed bridges are discussed below.

11.2.1 *Elastomeric Bearings*

Reinforced Elastomeric Bearings are simple yet versatile. These consist of steel laminates sandwiched between the Elastomer Layers. While the elastomer provides the ability to take rotation and translation by deflection, the ability to bear & transmit the Vertical Loads and Forces comes from the Steel laminates reinforcement.

Elastomeric Bearings follow simple configuration for transmitting the vertical loads, translation and rotation. The Elastomeric Bearings have moderate life span due to rubber material characteristics which is subjected to ageing under open environment as well as the wear due to regular movement and rotation. However, these bearings have the advantage of easy inspection and replacement, if required.

Elastomeric Bearings require larger space as they have low permissible design Stress as compared to mechanical Bearing systems. The restoring moment from the Elastomeric Bearings under peak rotation and movement conditions is required to be carefully analyzed while designing the associate components for e.g. the sliding interface combined with the Bearings. Elastomeric Bearings being vulnerable to ageing, ozone and fire may also need additional precautions. Experiences indicate that exceptionally large size elastomeric bearings require additional construction for their easy & safe lifting and handling without damaging the soft elastomer surface of the bearings.

Elastomeric Bearings may require special consideration for application in Cable Stayed bridges, as they are subject to considerable load fluctuations. Depending on their limitation to transfer high horizontal forces (such as those due to wind & earthquake, braking, etc) as well as rotations and translations, operating temperature etc, Floating Bearing configuration of elastomeric bearings may not be feasible. The arrangement of restraints or guide bearings in addition to Floating Elastomeric Bearings may provide a meaningful, long-lasting solution.

11.2.1.1 Anchored Elastomeric Bearings

The Elastomeric Bearings, generally, do not require positive Anchorages for fixing under normal bridge applications. However, for applications under dynamic loading conditions such as those in Cable Stayed Bridges, where the Elastomeric Bearing is susceptible to slippage out of position on account of extreme load fluctuations due to live load, wind and seismic, it may be appropriate to provide Elastomeric Bearings with positive Anchoring Devices. This would otherwise lead to slippage and dislocation of the bearings.

Anchoring of the Bearing can be effected in a most simple and cost-effective manner by means of an anti-skid plate (**Fig. 11.6)**, which is connected as an outer end plate to the bearing by effective vulcanization and fixed to the adjacent Structure by means of cementitious grout or epoxy bonding. However, for applications where this simple construction is not adequate and higher forces and extreme load fluctuations and vibrations are anticipated, more robust system of positive anchoring which includes top and bottom vulcanized plates and fixing to the adjacent structure by means of Bolts and Dowels shall be considered **Fig 11.7**.

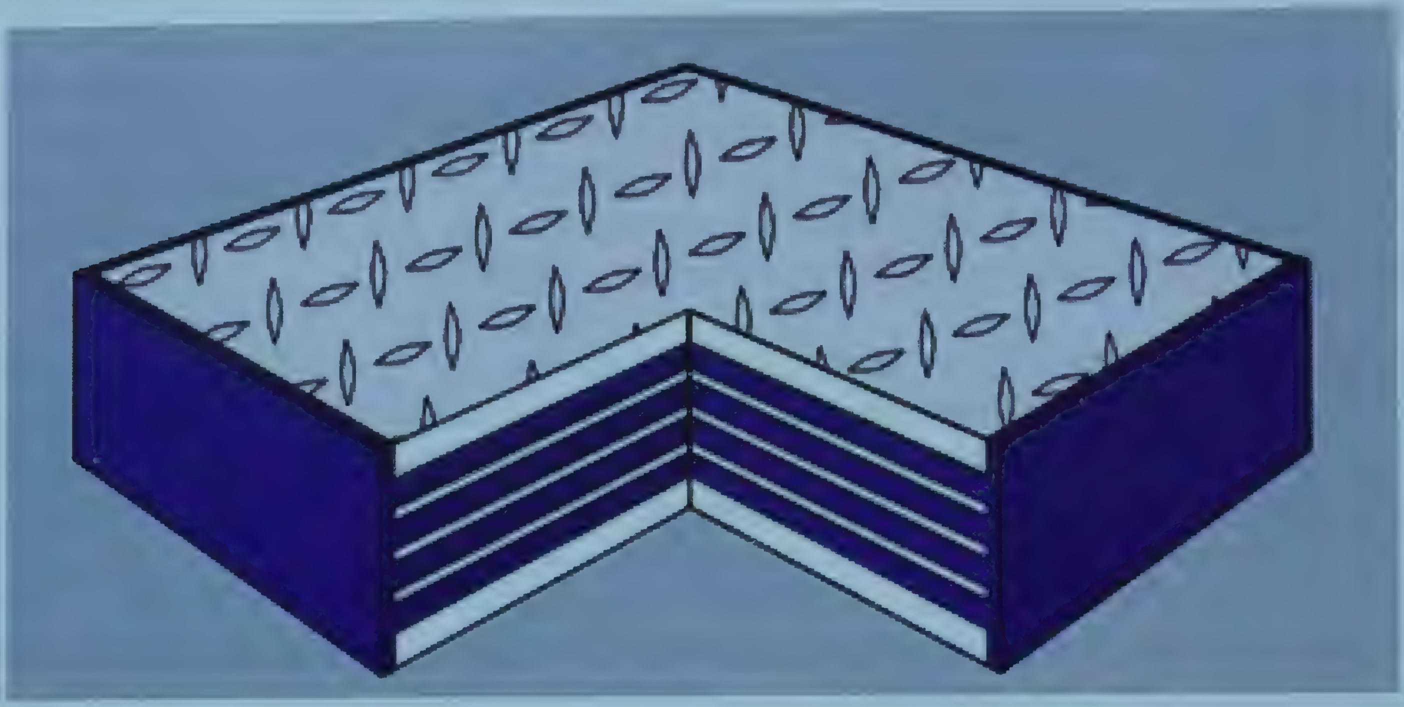


Fig. 11.6 Elastomeric Bearing with Anti-skid plate

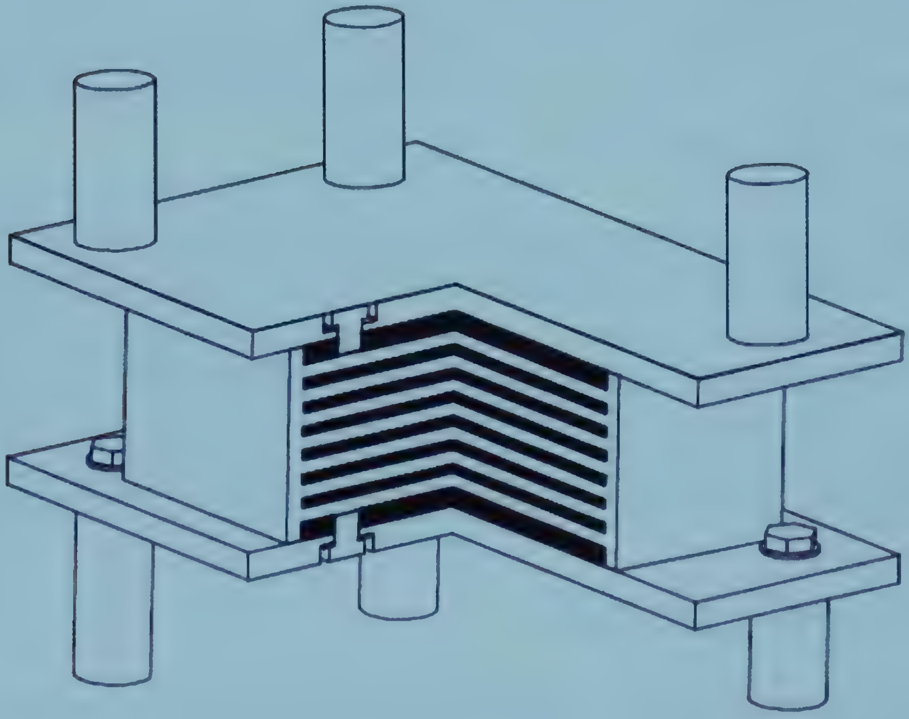
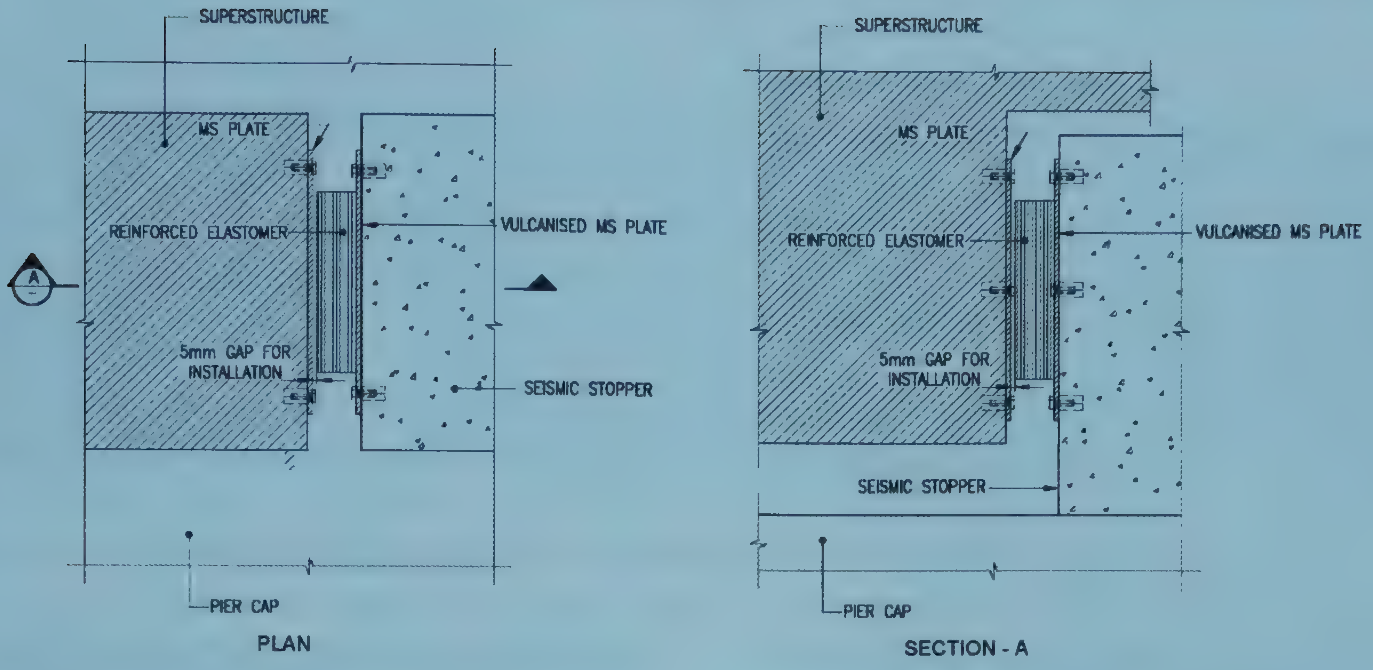


Fig. 11.7 Elastomeric Bearing with positive Anchorage

11.2.1.2 Lateral Restrained / Side Stopper Elastomeric Bearings

These Bearings are installed in Vertical position between the projections from Substructure and Superstructure (**Fig. 11.8 & 11.9**). They provide resistance to impact & horizontal forces to the structural members. Vertical Stopper Bearings can be of Fixed type as well as Sliding Type depending on the movement requirement at the location they are installed.

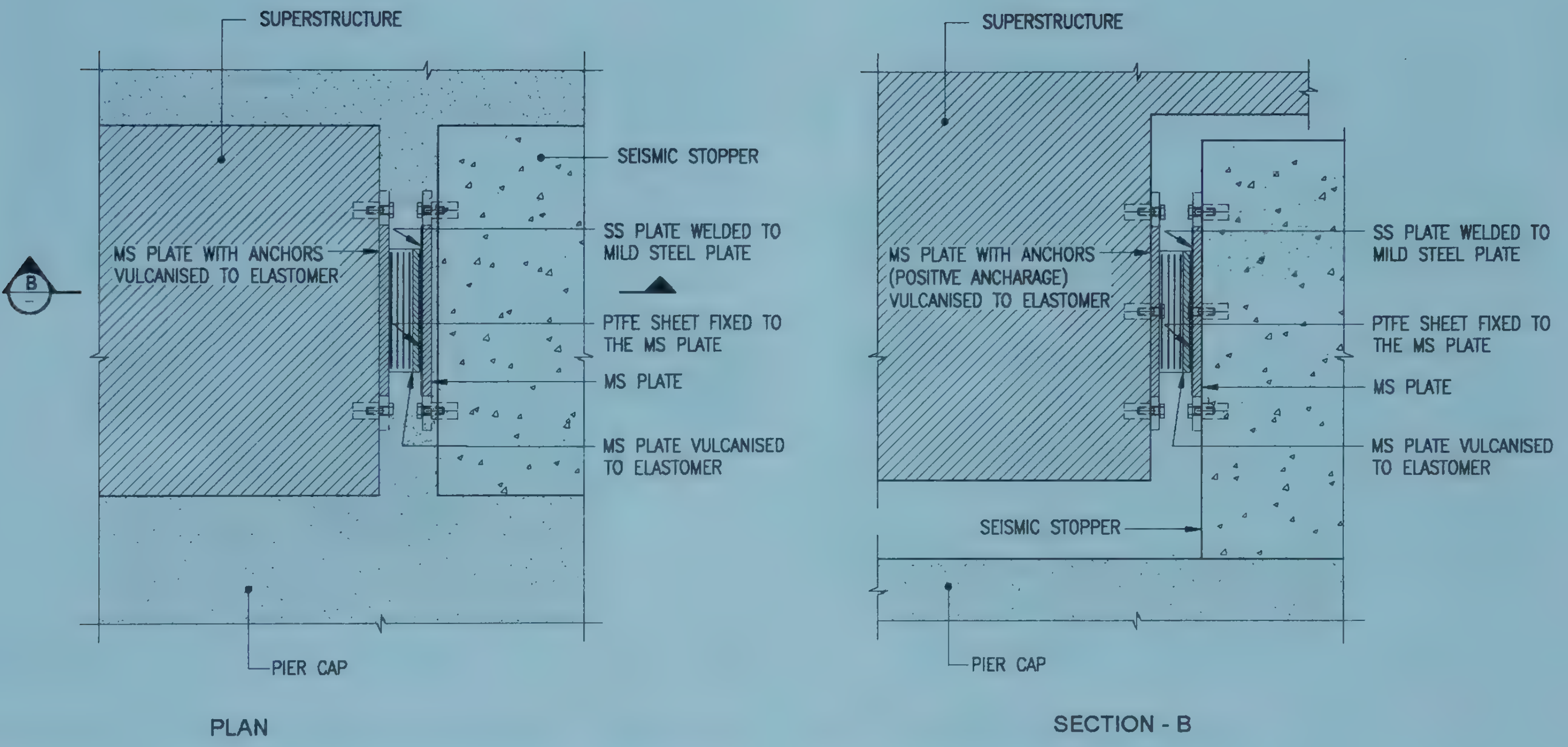
These Bearings are provided with vulcanized steel plates on one side for accommodating the positive anchoring devices.


SUPERSTRUCTURE
MS PLATE
REINFORCED ELASTOMER
VULCANISED MS PLATE
A
5mm GAP FOR INSTALLATION
SEISMIC STOPPER
PIER CAP
SUPERSTRUCTURE
MS PLATE
REINFORCED ELASTOMER
VULCANISED MS PLATE
5mm GAP FOR INSTALLATION
SEISMIC STOPPER
PIER CAP

PLAN

SECTION - A

Fig. 11.8 Seismic Stopper Elastomeric Bearing with positive Anchorage


SUPERSTRUCTURE
SEISMIC STOPPER
SS PLATE WELDED TO MILD STEEL PLATE
MS PLATE WITH ANCHORS VULCANISED TO ELASTOMER
B
PTFE SHEET FIXED TO THE MS PLATE
MS PLATE
MS PLATE VULCANISED TO ELASTOMER
PIER CAP
SUPERSTRUCTURE
MS PLATE WITH ANCHORS (POSITIVE ANCHARAGE) VULCANISED TO ELASTOMER
SS PLATE WELDED TO MILD STEEL PLATE
PTFE SHEET FIXED TO THE MS PLATE
MS PLATE
MS PLATE VULCANISED TO ELASTOMER
SEISMIC STOPPER
PIER CAP

PLAN

SECTION - B

Fig. 11.9 Sliding Type Seismic Stopper Elastomeric Bearing with positive Anchorage

11.2.2 *POT Bearings*

POT Bearings can offer freedom up to 4 degrees of movement and rotation. The pot bearing consists of a confined disk of elastomer within a short steel cylinder called POT. Loading is applied via a close-fitting steel piston. This puts the elastomer under high pressure, making it behave like a liquid, permitting rotation in any direction with very little resistance. The design allows high vertical loads. Via pot wall and piston large horizontal forces can be transmitted. (**Fig.11.10 & 11.11**). In the **Fig. 11.10**, (1) is the seal, (2) is PTFE and (3) is POT – Cylinder interface.

Fixed pot bearing offers transmission of Vertical Loads and Horizontal force while bearings equipped with a PTFE sliding surfaces allow free movable and guided design. Due to comparatively larger translational movements, importance must be given to testing for enhanced sliding path of the sliding components.

The performance and service life of Pot Bearings mainly depends on the effectiveness of the internal seal, intended to prevent the extrusion of elastomer that is under high pressure at the contact between the pot & piston.

The performance life of the POT Bearings largely depends on the service life span of the elastomer seal vis-a-vis accumulated rotation requirement at the bearing location which largely depends on the dynamic actions like live load deflection, temperature and wind induced rotation and deformations. Gapping effect at the sliding interface due to eccentric loading & rotation which is caused by elastomer stiffness is important for design consideration. Pot Bearings have limitation in satisfying large rotations. Specified limit of overall rotation is 0.03 radians and live load rotation limitation is 0.0025 radians unidirectional (for end support) and 0.005 radians (when installed at mid support of continuous span). Thus, due consideration is needed for Pot Bearings application in Cable Stayed Bridges where frequent & large magnitude rotation and dynamic movement situations are anticipated otherwise Pot Bearings may show signs of early maintenance and repair / replacement due to wear of Sealing Rings & Elastomer Pads.

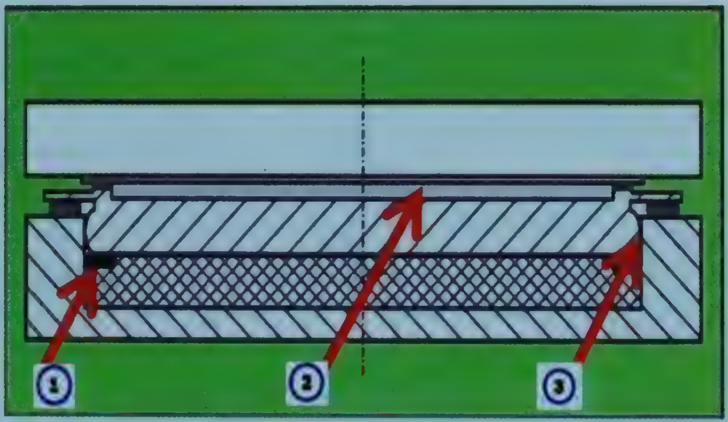


Fig. 11.10 POT Bearing Configuration

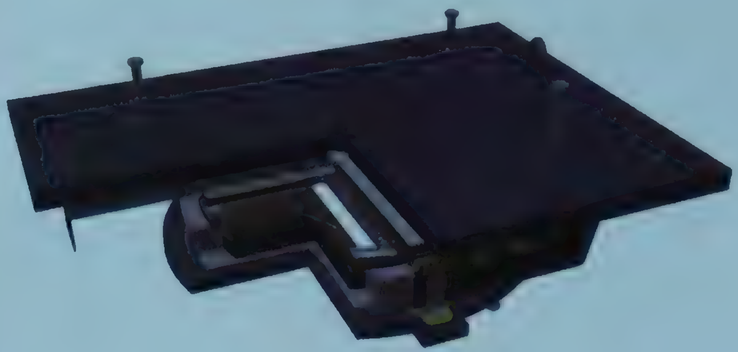


Fig. 11.11 Isometric View of POT Bearing

Further, due to the typical construction of POT Bearings adequate protection is needed at Pot-piston interface to prevent moisture / water ingress and standing water inside the POT due to gravity effects, to avoid corrosion. This corrosion between piston and POT would not only block the Bearing function but can also accelerate wear & tear of the seal.

11.2.3 *Spherical Bearings*

In this design, a spherical calotte corresponding to a concave lower part allows rotation about any axis (**Fig. 11.12, 11.13 & 11.14**). A flat sliding interface allows transition. The spherical bearing combined with PTFE and other improved sliding material has proven itself to function well. The bearing can be inspected easily in the installed state. The use of new modified sliding material not only enhances the accumulated sliding path capacity but also increased load carrying capacity over a smaller footprint as well as the ability to better accommodate the load variations that commonly exist in case of Cable Stayed Bridges.

The use of such modified sliding materials adds to the service life of the bearings and economy, especially, under large loads.

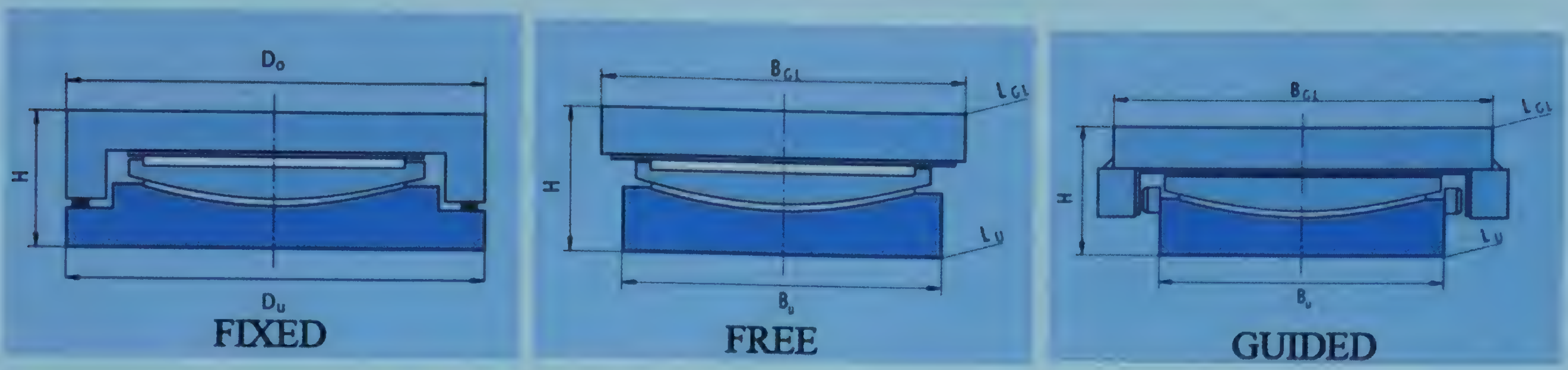


Fig. 11.12 Typical Configurations of Spherical Bearings

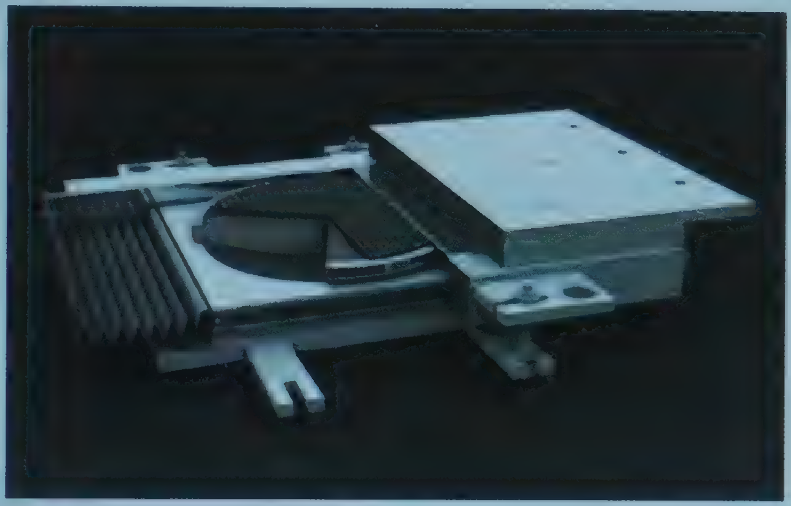
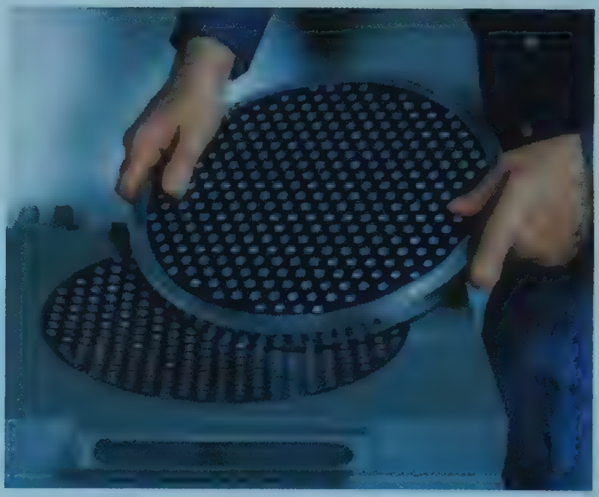
	

Fig. 11.13 Cut Section of Spherical Bearing	**Fig. 11.14 Concave - Convex Rotational Interfaces**

Due to comparatively large translational movements and high frequency dynamic rotation, importance must be given to testing for enhanced sliding path of the Sliding components. Also, care is required during their installation to avoid accidental disturbance to the level, alignment of the bearing and that of the superstructure resting onto the bearing prior to completion of superstructure skeleton. This is because Spherical Bearings unlike the Elastomeric and Pot Bearings can deflect easily with minimal amount of force.

11.2.4 *Anti-Uplift Bearings*

In Cable Stayed Bridges, often at the location where Back Stay cables are anchored to the deck or structural member holding the stays, bearing arrangement often requires anti-uplift consideration to resist occasional or permanent tension forces. Also, the combinations of varying loads and forces due to traffic, wind or seismic condition can create the requirement of uplift or tension forces which are to be resisted by such devices. **Fig. 11.15** depicts a typical Anti-uplift Spherical bearing.

It is worthwhile to mention here that options of handling the condition of Tension or Uplift forces by Structural Design consideration or by implementing special arrangements like Tension Bars or holding down cables and bolts are also worthy appropriate solutions. However, it is possible to design the bearings to handle such special conditions by themselves.

For the designing of Anti-uplift Bearings, due importance shall be given to whether the condition of uplift arising is only in ULS / Seismic or is in SLS also, i.e. day to day Service Condition. There is a need to differentiate between frequent uplift / tension or occasional Uplift as this affects the Fatigue Design requirements which are based, not only on the expected magnitude of load but also the corresponding number cycles during the intended Service Life period. Fatigue Design Considerations shall apply when designing the bearings for frequent Service Uplift conditions. Established literature and approved documents may be followed for the fatigue design.

Challenges in Design and Production:

i) The uplift forces including applicable horizontal movements shall be carried safely by the Bearing in any service and ultimate load condittion.

ii) The Bearing system shall not develop significant Vertical Play / Gap in uplift condition which may otherwise cause fatigue effects due to impact loading and rattling of the components in load reversal conditions.

iii) All components of the Bearing must be capable of carrying adaequate number of load cycles without fatique.

iv) Lifetime, friction and restoring moments shall not be restricted by the uplift design.

v) Since the standard Bearing design rules may not be adequate for design of Uplift Bearings, special design considerations and FEA analysis may be used for design verification.

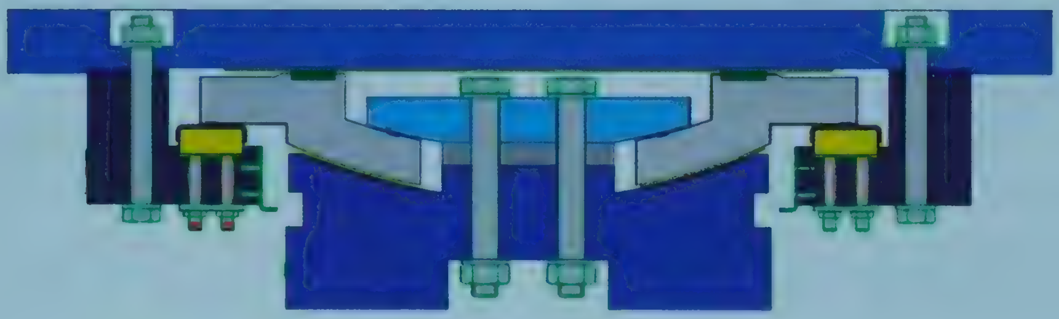


Fig.11.15 Typical Anti-uplift Spherical Bearing

11.2.5 *Tension Bearings*

Unlike the Anti-uplift Bearings which have to resist occasional uplift under load reversal conditions, the Tension Bearings are those which are deployed at the Locations where a permanent tension force exists simultaneous to requirement of freedom for translation and rotation. Such applications are common at the location of back stay cables anchoring to deck or at anchor locations in a cable stayed Bridge where permanent tension forces might occur. This requirement can be dealt with by providing long pre-stressed anchor bars freely suspended and fixed with spherical end plates and nuts or by pendulum system with spherical hinge connection at both ends for providing rotation as well as limited sliding freedom. For enhanced displacement capability, the design of pendulum Bearings can be modified with sliding interface while ensuring no separation at sliding surface under all conditions of loads and movement. It is important to ensure the transfer of huge tension forces deep within the substructure by means of long anchor bars thereby satisfying the local pull-out failure condition, **Fig. 11.16 (a)**.

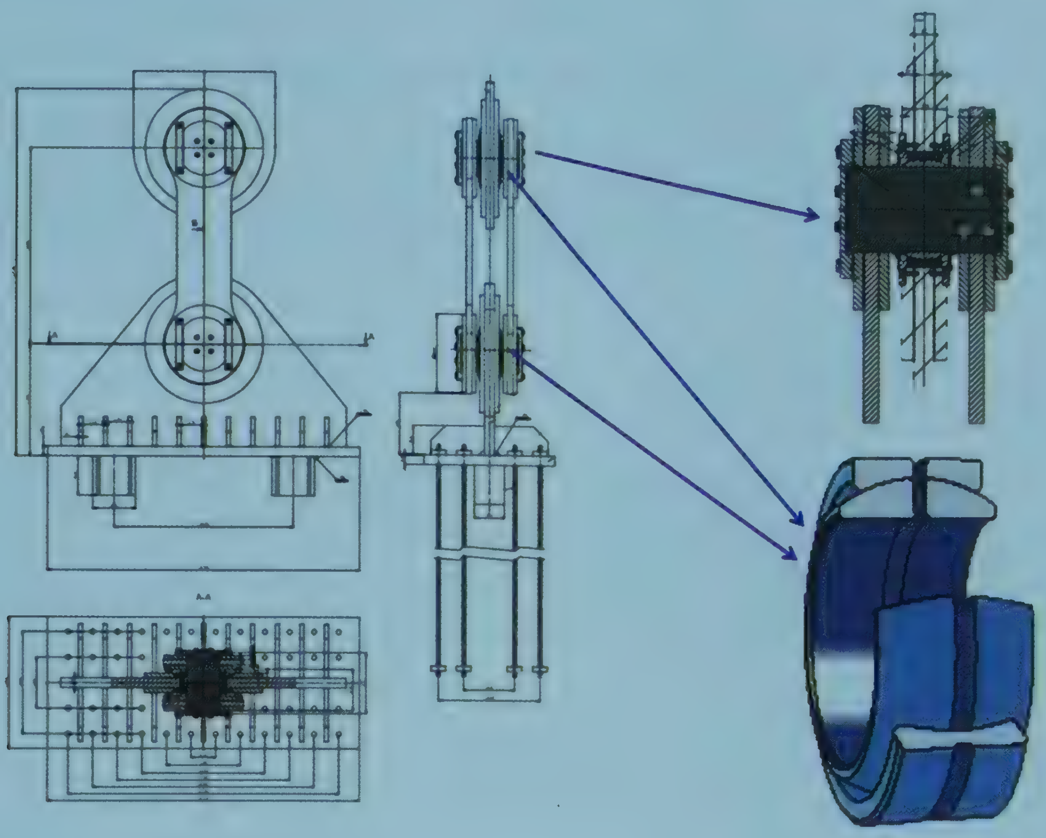


Fig. 11.16 (a) Typical Sketch of Pendulum Tension Bearing with Deep Penetrating Anchors

Signature Bridge in Delhi has employed such Pendulum Tension bearings. **Fig 11.16 (b)** depicts the bearing for tensions coupled with seismic forces. Design of Such Bearings also requires sufficient experience and considerations similar to Anti-uplift Bearings. Approved specialist literature may be followed for the purpose.

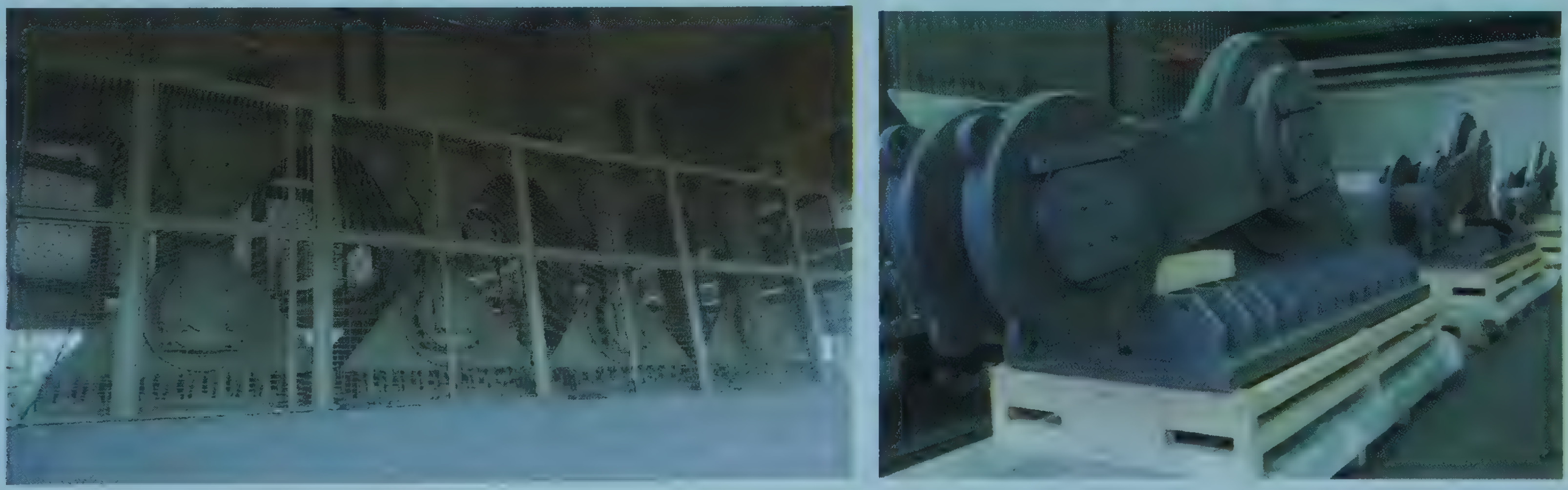


Fig. 11.16 (b) Pendulum Tension Bearing used in Signature Bridge, Delhi

11.3 Expansion Joints

Movement Joints commonly called as Expansion Joints are proprietary products which are classified into different types, primarily based on their movement capacities. As per the IRC:SP:69 Specifications, the maximum gap opening is limited to 80 mm, thus up to 80 mm total movement requirement, single strip seal expansion joint may be used. For requirement of movement in excess of 80 mm, multi-module large movement Expansion Joints like Modular Strip Seal type shall be required.

For application in Cable Stayed Bridges, it is desirable that the expansion joints possess flexibility to accommodate structural movements and differential deformation or rotations about all axes X, Y and Z. In other words, the expansion joints shall possess all six degrees of freedom.

The traffic volume on any important cable stayed bridge is significant and may easily reach 100000 vehicles per day or even more. Therefore, the joint design must especially account for fatigue and wearing design.

11.3.1 *Single Strip Seal Expansion Joint*

Single strip seal expansion joints consist of two heavy duty steel edge beams with anchorages in the form of Shear Headed Studs or Plate with Loop anchorages and an elastomeric sealing element specially shaped to lock mechanically inside the cavity of steel edge Beam. This mechanical locking of the elastomer seal not only guarantees the water tightness of the joint, it also ensures that the system integrity is maintained even at extreme opening and at differential movement as well as rotation condition in the structure. **Fig.11.17** shows a typical Single Strip Seal Expansion Joint.

The maximum gap width during opening of joint between the edge beams at road surface shall be limited to 80 mm for safe passage of' the traffic. The heavy-duty edge beams protect the adjacent bridge deck concrete from damage due to vehicular impact and also transfer the vehicular load to the deck structure through robust anchorage system.

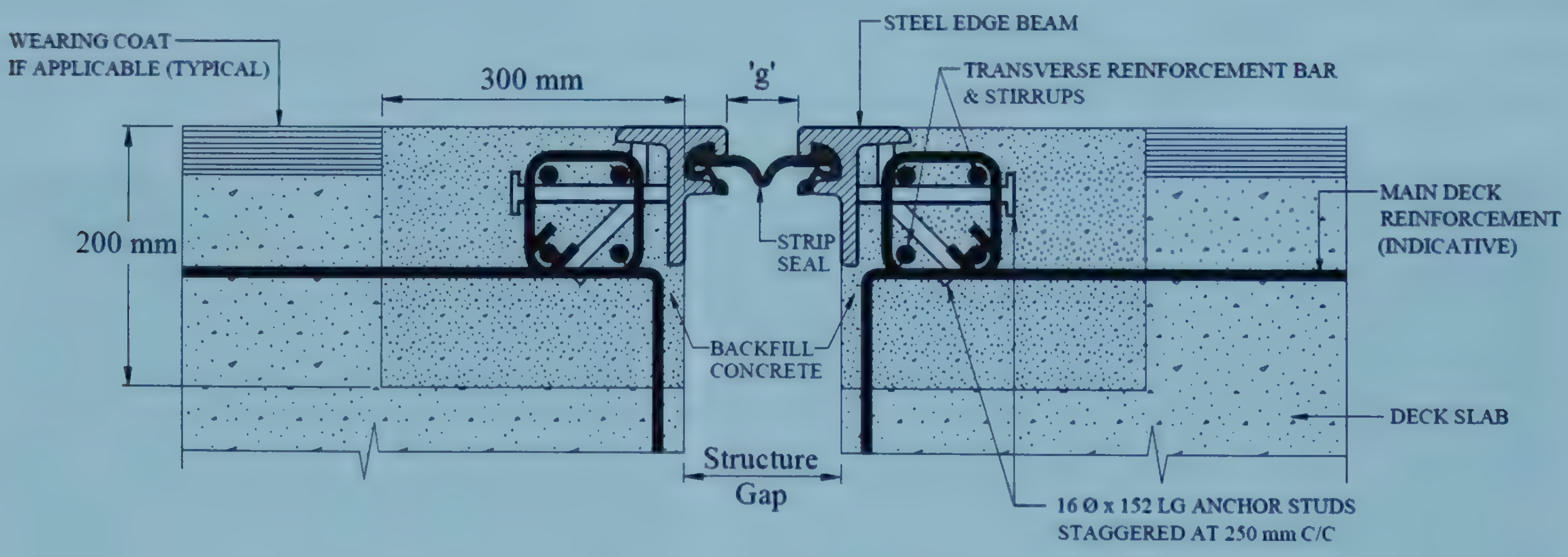

WEARING COAT
IF APPLICABLE (TYPICAL)
300 mm
'g'
STEEL EDGE BEAM
TRANSVERSE REINFORCEMENT BAR
& STIRRUPS
200 mm
STRIP
SEAL
MAIN DECK
REINFORCEMENT
(INDICATIVE)
BACKFILL
CONCRETE
DECK SLAB
Structure
Gap
16 Ø x 152 LG ANCHOR STUDS
STAGGERED AT 250 mm C/C

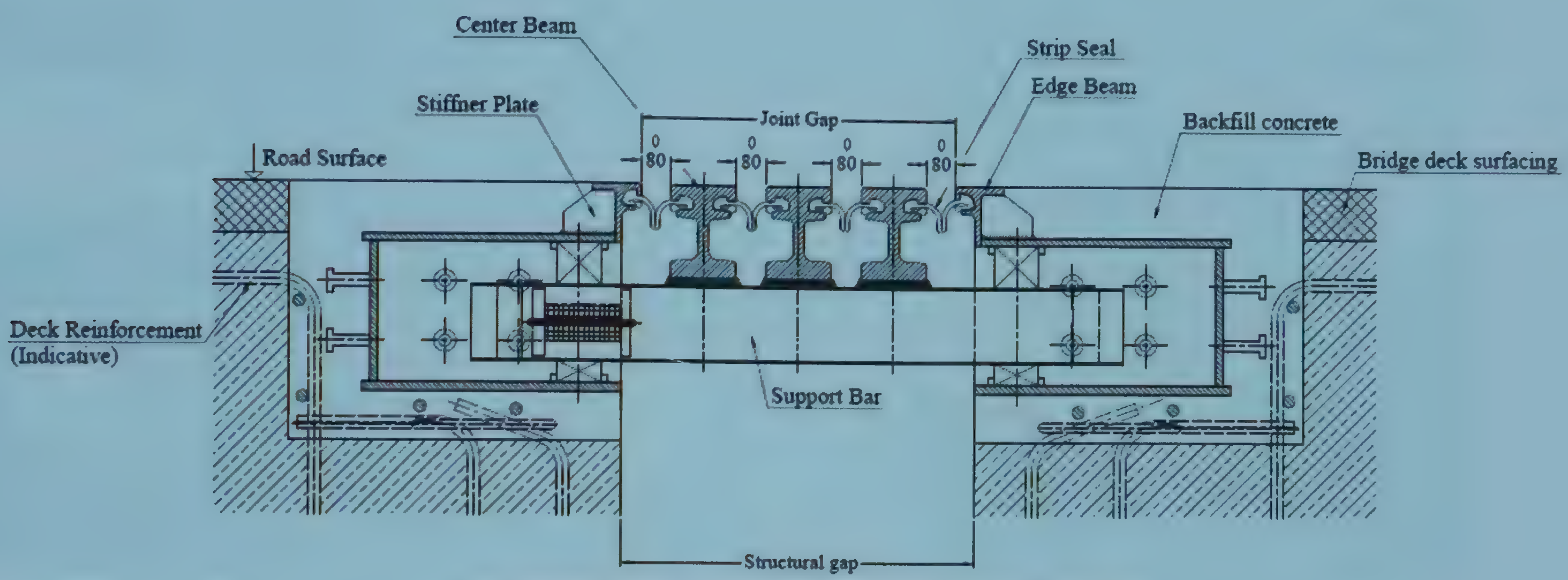
Fig. 11.17 Typical Single Strip Seal Expansion Joint

11.3.2 *Modular Strip Seal Expansion Joint*

The Modular strip seal expansion joint divides the total movement capacity into several smaller gaps. Centre beams orientated transverse to the traffic direction form the load-carrying elements and are supported by transverse support beams. Centre beams and transverse support beams are connected by assembly of steel stirrups holding the elastomeric support elements transferring the traffic-load into the bridge structure.

Geometrical or elastic gap control systems ensure equal opening & closing of all gaps under different movement conditions. **Fig. 11.18** shows a typical Modular Strip Seal Expansion Joint. In case of high seismic movements, these expansion joints may be replaced by the ones indicated in **11.3.3**.

The joint is made watertight through elastomeric strip seals which are mechanically locked inside the cavity of Steel edge and Centre beams. The modular Joint system especially for application in flexible structures like Cable Stayed and Suspension Bridges shall be able to accommodate free movement and rotation in all three axis directions.



CROSS-SECTION OF MODULAR EXPANSION JOINT SYSTEM

Fig. 11.18 Typical Modular Strip Seal Expansion Joint

In order for the Modular Joints to accommodate the multi-direction movement in a Cable Stayed Bridge, the Modular Joint systems having three-dimensional movement capability with flexible connection between the Centre Beams and Support Bars shall be preferred.

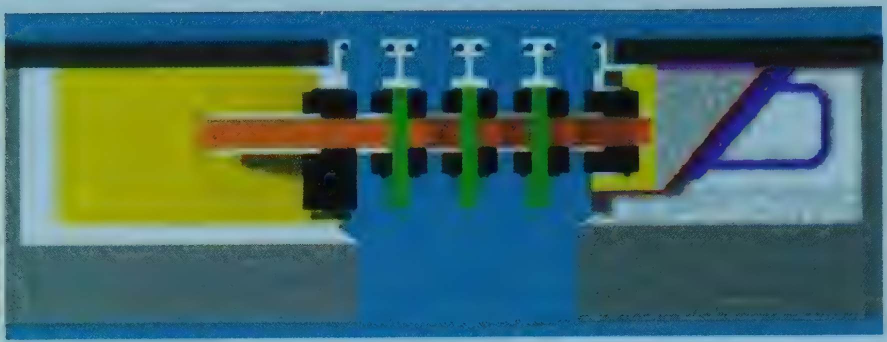
11.3.3 *Modular Joints to cater for large Seismic movements*

Seismic Joints either accommodate seismic displacement by means of modular lamella systems or by a special Fuse Box System (predetermined breaking point device). With the help of these two systems, damage to the Structure can be prevented or limited. Roadworthiness and crossing are ensured during and after the earthquake.

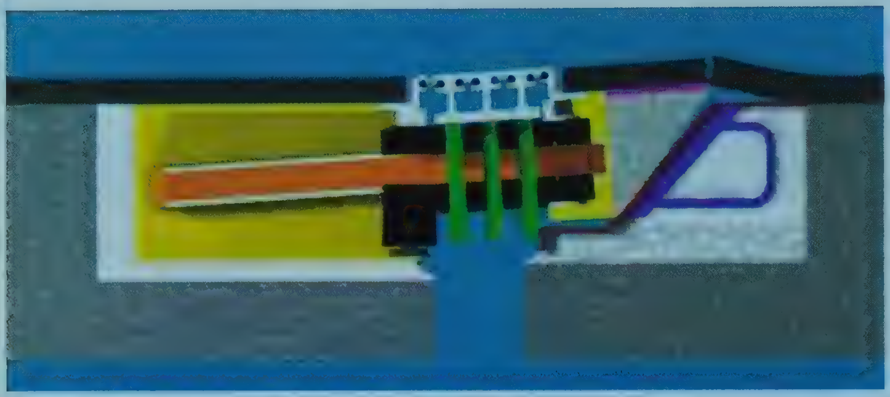
11.3.3.1 Modular Joints with Seismic Fuse Box

Modular Joints equipped with Seismic Fuse Box System **(Fig. 11.19)** allow for a more cost-efficient design of the expansion joint; it does not actually avoid but limit the possible damage at the Expansion Joint location. Fuse Box Systems permit constructive yielding of the bridge closing movements up to the anticipated threshold state (ULS) and possibly even beyond it. Depending on the Fuse Box System, the resulting closing movement is absorbed, either through an oblique-angled, upwards-deflecting plane of disengagement (fuse connection) or through a vertical lowering of the construction with subsequent sliding in the joist box. The Fuse Box System protects the deck of the bridge from excessive loads and destruction. Emergency crossing of rescue services over the expansion joint after the earthquake, once seismic fuse is activated, may be possible with caution and temporary measures. Repair of the Seismic Fuse construction and its adjacent road surface will be required, though.

Seismic Fuse Modular Joint in Production



Modular Joint with Seismic Fuse in normal working condition



Modular Joint with Seismic Fuse Activated after an Earthquake

Fig. 11.19 Modular Joint with Seismic Fuse Box

11.3.3.2 Modular Joints with large Seismic movement capability

The Seismic movement modular Joint (**Fig. 11.20)** is an enhancement of the earlier Modular Strip Seal Expansion Joint, adding, considerably, to the range of application where large and complex movements are anticipated caused by the reasons associated with geometry, movements due to earthquake, , or applications in restricted space, the use of conventional Joints is not suitable. This type of joints are able to take large seismic movements and remain fully functional and shall not require any restoration after the seismic activity.

This type of Joint is suitable for variable deformations along the bridge structure. The joint not only follows the main movement of the bridge in carriage-way direction but also distinctive movements in the 2 spatial directions perpendicular to the main direction. Even rotations of the bridge about the three spatial axes are easily borne, thereby making these Joints virtually limitless to cope with any direction and magnitude of movement and rotation.

The centre beams of the Seismic Modular Expansion Joint can slide on obliquely arranged swiveling support bars with the help of sliding bearings. Lift-off from the sliding bearing is prevented by means of a prestressed sliding spring that is arranged in the support stirrup underneath. Stirrups provide constant prestressing that is geometrically controlled.

By means of this geometrical arrangement of the support bars, the position of the centre beams is controlled such that the overall width of the joint opening is equally subdivided into the joint gaps between the centre beams and between the centre beams and edge beams, respectively. This simple and effective control mechanism results in unrestrained absorption of movements without any defined direction and simultaneous transmission of traffic loads safe guarding additional control elements.

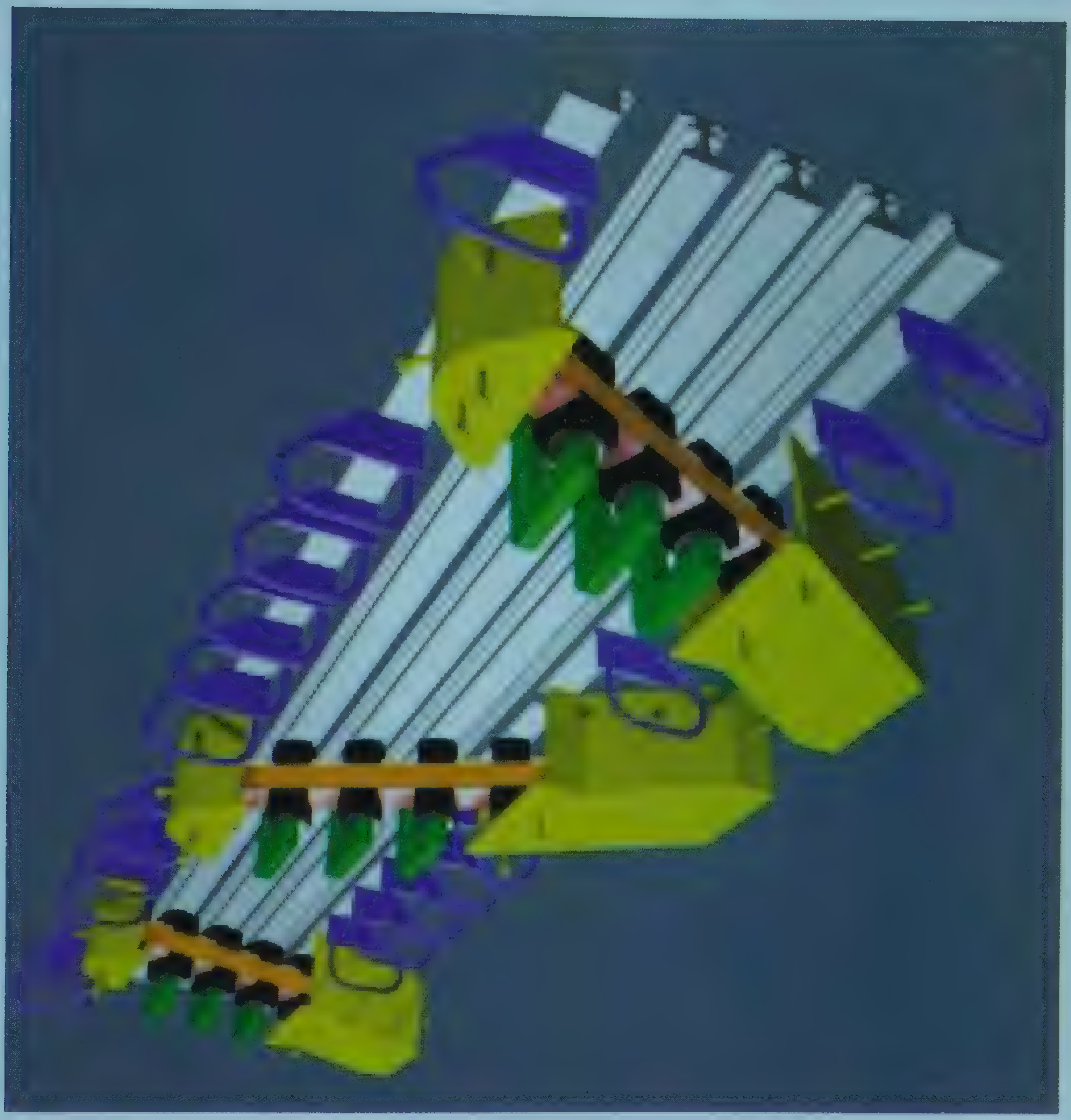


Fig. 11.20 Typical Large Seismic Movement Modular Joint construction

11.3.4 *Detailing of Expansion Joint in Crash Barrier*

At Crash Barrier Locations where Joint Gaps are small say 80-100 mm, the Expansion Joint may be provided with a simple oblique upturn so as to prevent the rain water falling underneath from the Joint end. However, for large gap opening especially in cases were lateral movements are also anticipated, leaving such large openings at the Crash Barrier location uncovered could lead to safety concerns.

Hence for large Gap opening at Crash Barrier where no significant lateral movement occurs, the gap shall be either covered with a metal cover plate system having freedom to slide and bridge the variation in Gap due to thermal movements.

Alternatively, the modular Expansion Joint may be designed to follow the Crash Barrier profile (**Fig. 11.21**) and installed accordingly to run up to the full height of the Crash Barrier.

For large to very large Gap openings especially where significant lateral movement is also anticipated, for instance in Cable Stayed Bridges, it is also a practice to provide dummy Steel construction on top of the Modular Expansion Joint Centre Beams that follows the Crash Barrier Geometry so as to maintain the aesthetics while covering the Gap opening.

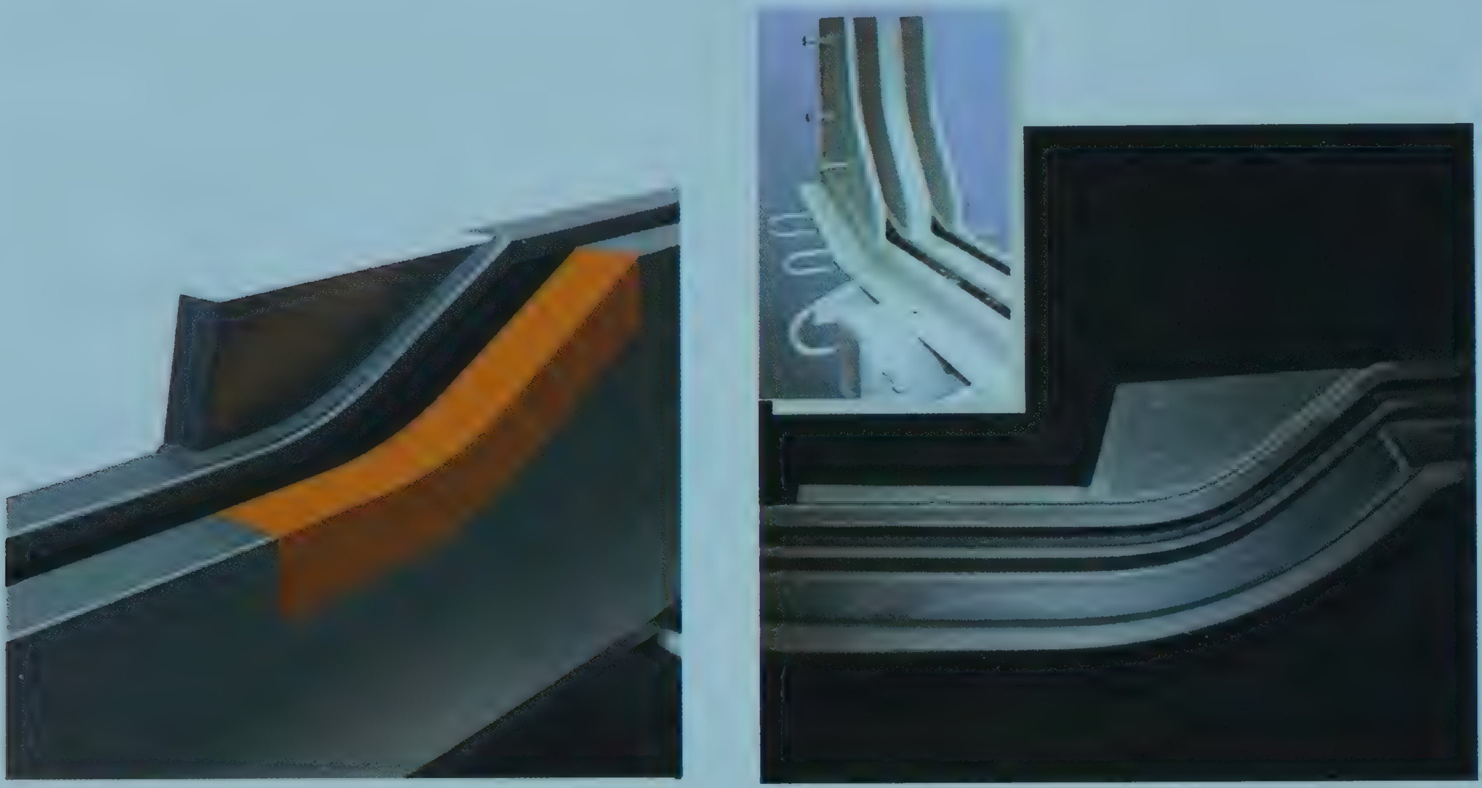


Fig. 11.21 Detailing of Expansion Joint at Crash Barrier Location

11.4 Seismic Protection Devices

11.4.1 Seismic Isolators

When the Bearings, in addition to satisfying the basic feature of transferring Vertical Load, are also required to fulfill the task of Isolation and energy dissipation under Transient Seismic events, special Isolation Bearings may be utilized for the purpose.

There are 4 main functional requirements of Seismic Isolators:

i.) Transmission of vertical loads

ii.) Lateral flexibility

iii.) Re-centering capability

iv.) Energy dissipation by Damping

Isolation Bearings are basically of two types:

- i.) Elastomeric Isolators
- ii.) Mechanical Isolators: Sliding Isolation Pendulum Bearings (SIPS)

11.4.1.1 Elastomeric Isolators

Elastomeric Isolators finds their field of application both in Buildings as well as Bridges. They offer economical solution, additional damping is often necessary as the damping of the elastomeric bearings, even with high damping elastomer bearings is quite low. Although they do not suffer damage in case of design earthquake, sometimes have problems with fire resistance.

Elastomeric Isolator Bearings (Usually Lead Rubber bearing wherein low damping elastomer is used in combination with Lead Core) is often found a reasonable and effective solution to be used in small span cable stayed bridges as support cum isolation system, thereby mitigating the risk of damages due to earthquake and wind forces. **Fig. 11.22** shows some typical Rubber Isolators.

Elastomeric Isolators with lead core increases its damping properties. While the lead core offers high damping by way of plastic deformation, the re-centering is still achieved by elastomer itself. The stiffening effect of Lead Core against the Elastomeric restoring capacity is to be considered. Purity of Lead Core (99.9%) is extremely important.

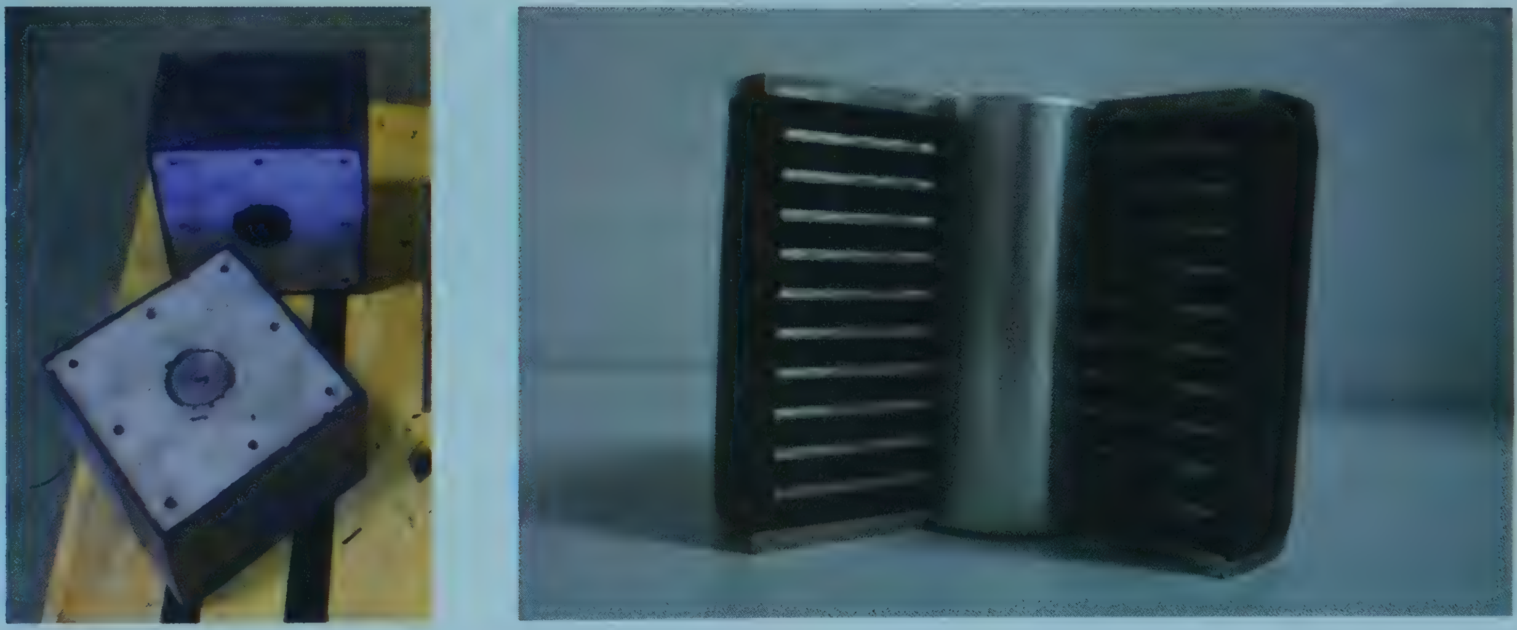


Fig. 11.22 Typical Types of Rubber Isolators

Elastomeric Isolators are more suitable for application where the Seismic effective periods are comparatively small and also the maximum seismic displacements and peak Load values are less.

11.4.1.2 Mechanical Isolators (SIPS)

Mechanical Isolators (**Fig. 11.23)** in the form of curved surface slider or pendulum Bearings rely on sliding friction and pendulum principle, respectively, to offer the desired load transfer, isolation, energy dissipation and re-centering behavior. A reference may be made to IRC:SP:114 for the same.

While the SIPS can be used both for non-seismic as well as seismic isolation, they can offer elongation of effective period far beyond the range of Elastomeric Isolators which can be combined with large rotation and displacements of say +/- 1000 mm or more. Since, no separation at the sliding surfaces occurs, no significant wearing takes place,

In SIPS unlike the Elastomeric Isolators, the time period is totally independent of the Load condititon and can be precisely defined by selecting the required radius of the curved surface. Re-centring capacity and energy dissipation / sliding behaviour also does not depend on load, ageing or even temperature as in the case of Elastomeric Isolators.

SIPS can be designed for extremely high loads, yet offering smaller footprints by adopting the telescopic double curved slider geometry with no spring effects as compared to Elastomeric Isolators. Direct steel-steel-contact can also be avoided by utilizing superior sliding materials as per the relevant bearing codes resulting in service life time of 50 years and longer.

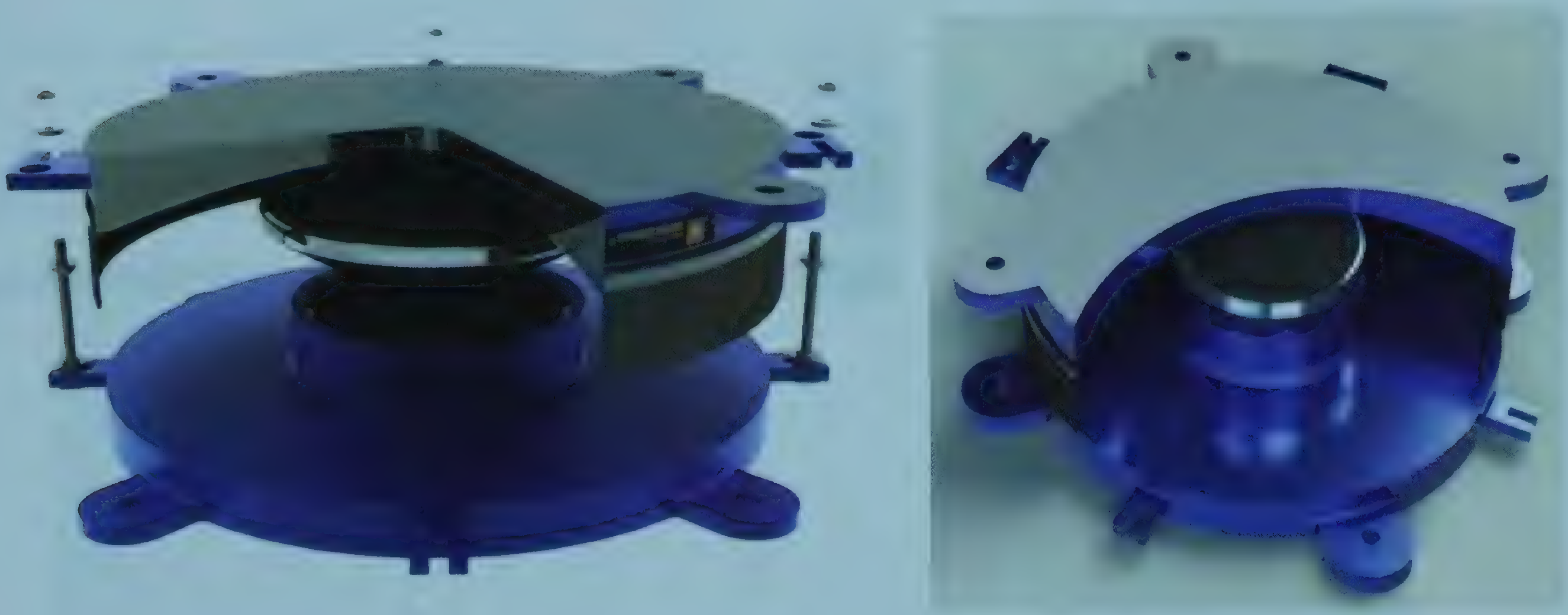


Fig. 11.23 Mechanical Isolators

11.4.2 *Dampers & STUs*

11.4.2.1 Hydraulic Dampers

Hydraulic Dampers (**Fig. 11.24)** are velocity dependent devices. They allow slow movements typically (< 0.1 mm/s) without appreciable resistance, say, under 5% of design force, but resist sudden displacement (due to service force impacts like wind, traffic or that due to Seismic) without appreciable deformations (0.5 - 3% of stroke capacity in loaded direction). When overload they are able, depending on their design, to dissipate energy for seismic load cases.

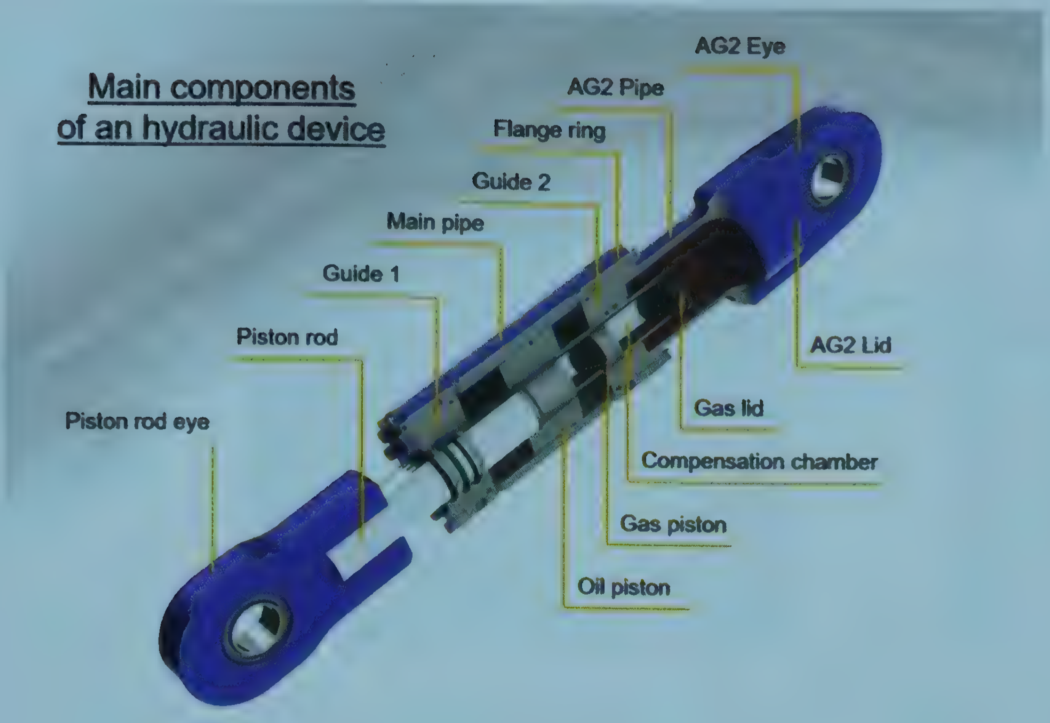

Main components of an hydraulic device
AG2 Eye
AG2 Pipe
Flange ring
Guide 2
Main pipe
Guide 1
Piston rod
Piston rod eye
AG2 Lid
Gas lid
Compensation chamber
Gas piston
Oil piston

Fig. 11.24 Hydraulic Damper

Advantages of Hydraulic Devices

1) It can offer efficient energy dissipation with a variety of hysteretic loops.

2) Cost reduction resulting from reduction in seismic demand.

3) Leak free due to very low pressure during the operating period & triple sealing

4) Damping exponent available from as low as 0.04 and upwards up to 2 to suit the Structural demands.

5) Short response time due to low compressibility.

6) Behavior independent of Temperature variation.

7) Lock-up of service impacts

8) Design considers even service load cycles for fatigue

9) Accommodation of accumulated sliding path of 50 km or more possible.

Hydraulic devices, in general, can be distinguished in three main types:

i.) Shock Transmission Unit (**STU**)
ii.) Shock Transmission Units with load limiter (**STL**) and
iii.) Hydraulic Damper (**HD**).

While at low velocities the behaviour of all the three devices are the same, at high velocities the behaviours differ from each other (**Fig. 11.25**). All devices can work in a regular range of - 40 to 70°C, which can also be adjusted on demand. General principles of working of Hydraulic Devices are enumerated in **Fig. 11.26**.

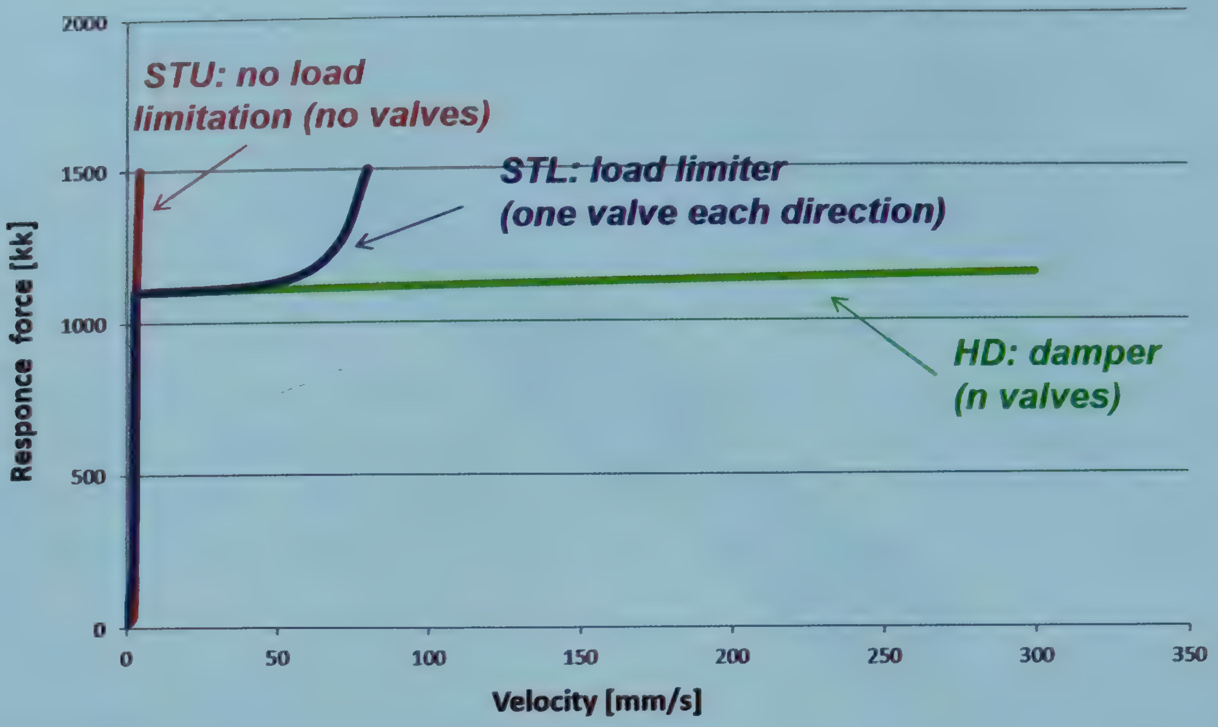


Fig. 11.25 Behavior of Various Hydraulic Devices

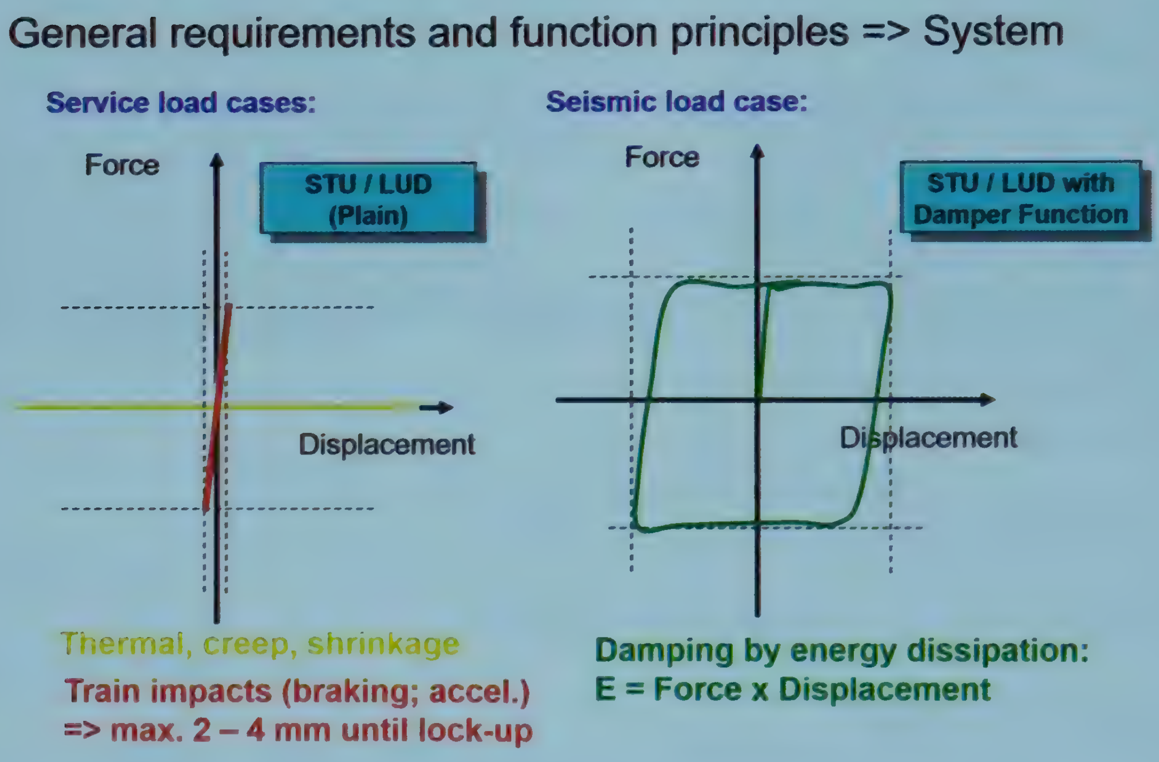

General requirements and function principles => System
Service load cases:
Seismic load case:
Force
STU / LUD (Plain)
Displacement
Thermal, creep, shrinkage
Train impacts (braking; accel.)
=> max. 2 – 4 mm until lock-up
Force
STU / LUD with Damper Function
Displacement
Damping by energy dissipation:
E = Force x Displacement

Fig. 11.26 General Principles of Hydraulic Devices

All Hydraulic devices, irrespective of their types, shall be provided with a Spherical Hinge **(Fig. 11.27)** for Pin connection so as to facilitate minor misalignment or limited lateral rotation / displacement without hampering its function. The typical value of misalignment that can be handled with this mechanism is usually limited to 4 degrees.

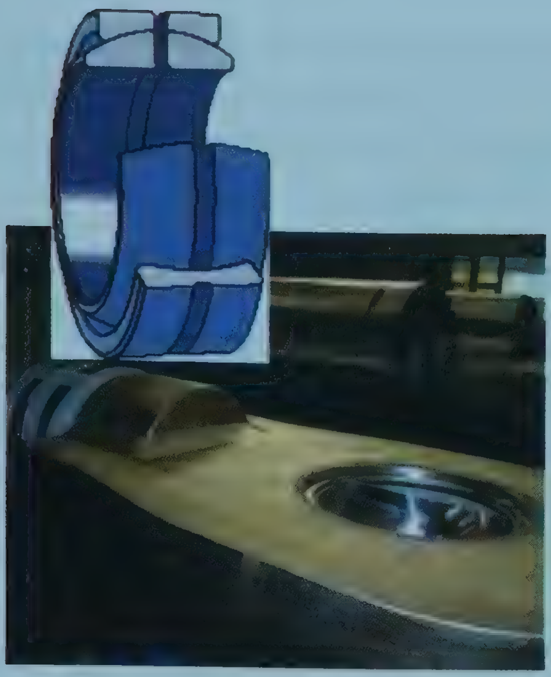


Fig. 11.27 Hydraulic Devices: Spherical Hinge

11.4.2.2 Shock Transmission Unit (STU)

Low velocities displacements of the structure caused by thermal actions, creep and shrinkage, are accommodated without appreciable resistance (maximum 10% of the nominal force) by the STUs. This behaviour aims at allowing free slow acting movements, without inducing undesired forces acting on the structure.

For higher velocities of the structure the device locks up starting typically from 1 mm/s and transmits longitudinal dynamic actions without appreciable displacements. The devices exhibit low elasticity levels, about 2-5% of the stroke. Scope of the hydraulic shock transmitters in this phase is that of creating additional temporary restrains under short-term dynamic actions (longitudinal seismic actions, strong braking or wind gust) in order to distribute the force to the supports other than the restrained pier/abutment or predetermined braking points for better distribution. **Fig. 11.28** for fixing typical arrangement

If the velocity raises beyond the design input, the response force increases up to failure of the device, its connection or the Structure itself.

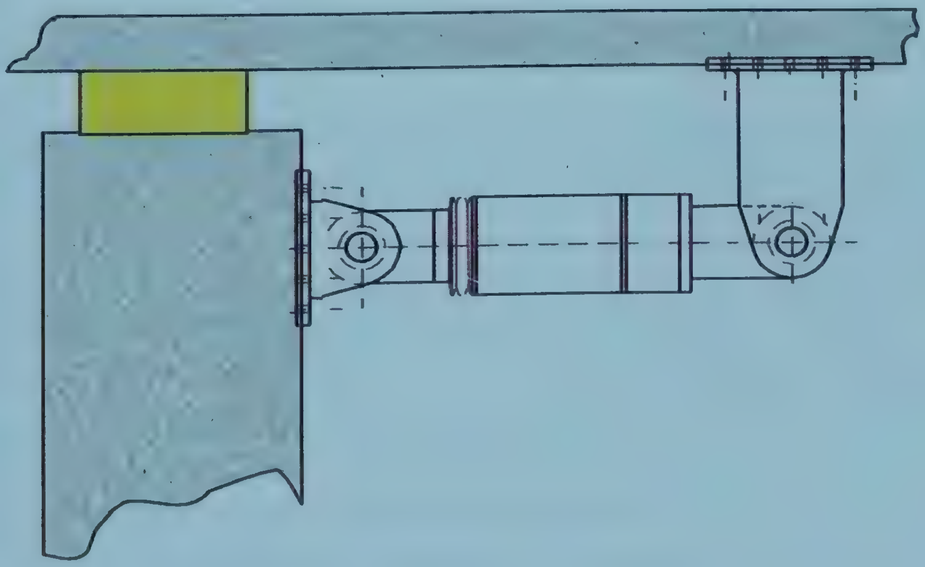


Fig. 11.28 Typical Fixing Arrangement of STU

In the shock transmission units, both resistance to the movements due to thermal variations and deformations consequent to an earthquake attack have been minimized, the velocity V_0 is usually individually adapted in the range from 0.1 to 1.5 mm/s, but for very large structures can reach the value of 5 mm/s.

11.4.2.3 Shock Transmission unit with load Limiter (STL)

Due to the fact that tolerances exist:

a) within the structure,

b) within stiffness of each device,

c) within the occurring lock-up velocity of each device, and

d) within the STU-bracket system

the parallel and serially arranged STU's will not get activated simultaneously. Instead, each one of them will develop a different response force.

In case the STU is designed without force distribution function, the difference in tolerances effect is a considerable higher input force on some devices. For this reason, specifications require STU to be designed considering 150% of the rated design force. Moreover, this increase in design force is not only applicable for the STU device but for the support brackets, the anchor system and finally for the structure itself, too. Yet, this increase in input force for design may or may not be even sufficient and the overall system is still not fail-safe.

An STL (**Fig. 11.29)** on the other hand displays the same working principle as an STU, but offers the possibility of a load limitation. One valve in each direction opens after reaching a predetermined pressure level, the device yields causing a movement of the structure. Thus, the transmitted longitudinal forces are limited to a desired value in the case unexpected seismic actions exceeds those resulting from the design level earthquake. This option aims at avoiding damages to the structure (piers or abutments), anchoring devices, and the shock transmitter itself.

STL are also used to ensure that all shock transmitter in a structure activate together.

The load is limited up to a certain velocity. For higher velocities the valve section is too small to allow a fluid flow Q = v*A (continuum equation) through the chambers, and the force begins to rise again.

The reliability factor of shock transmitters on their design force shall be $\gamma x=1.5$, unless an overload protection system or "load limiter" is incorporated. In this case, the value of the reliability factor can be reduced to $\gamma x=1.1$ and shall be applied to the design system force F_0 specified by the designer. The adoption of STL decreases the forces acting on the structural members by say 26%. It increases the overall safety of the devices and the structure as it is granted that all devices in serial and parallel arrangement are equally and simultaneously loaded when affected by sudden service or seismic impacts. This is not the case with classic STUs. These might be overloaded even with more than the reliability factor of 1.5 applied onto the design force. Therefore the STL application reduces costs of the structural members and even cost of the shock transmitter itself, because an STL is more compact, i.e. smaller than an STU. The STL is an economical solution, while providing additional technical benefits and reliability.

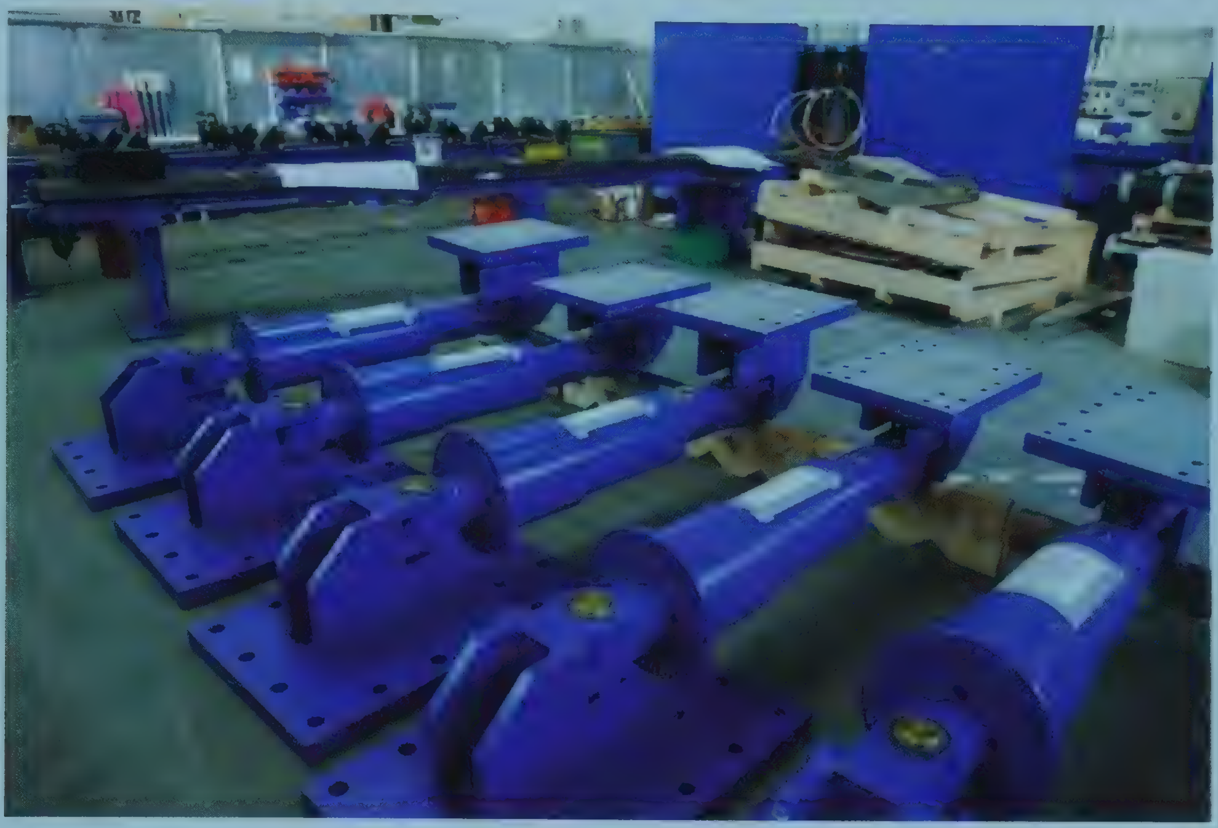


Fig. 11.29 Shock Transmission Unit (STU)

Advantage of a Shock Transmitter with Force Limiter (STL)

i) STUs without overload protection require a safety factor of 1.5 (pr EN15129, chapter 5.3.3)
ii) STUs with overload protection need only a safety factor of 1.1
iii) This means that adjacent structures can be designed for a 26 % less response force when an overload protection is incorporated into an STU.

Correspondingly, savings in the adjacent structure can be realised

11.4.2.4 Hysteretic Dampers

The Hysteretic Dampers (**Fig. 11.30)** unlike the Hydraulic Devices are displacement dependent devices. These safety devices are more often used in Building Construction but also finds their application in bridge constructions. Usually, Steel-Hysteresis-Dampers are installed in combination with re-centering elements, e.g. elastic columns, elastomeric bearings or sliding isolation pendulum bearings etc.

Hysteretic Dampers exhibits the following Advantages:

i) Higher initial stiffness for small displacements

ii) Re-centering might be not always sufficient and might be enforced by additional devices

iii) Max. force at max. displacement

iv) More effective displacement control possible, compared to bearing/isolator system alone.

v) Highly efficient energy dissipation

vi) Temperature independent

vii) Aging- resistant

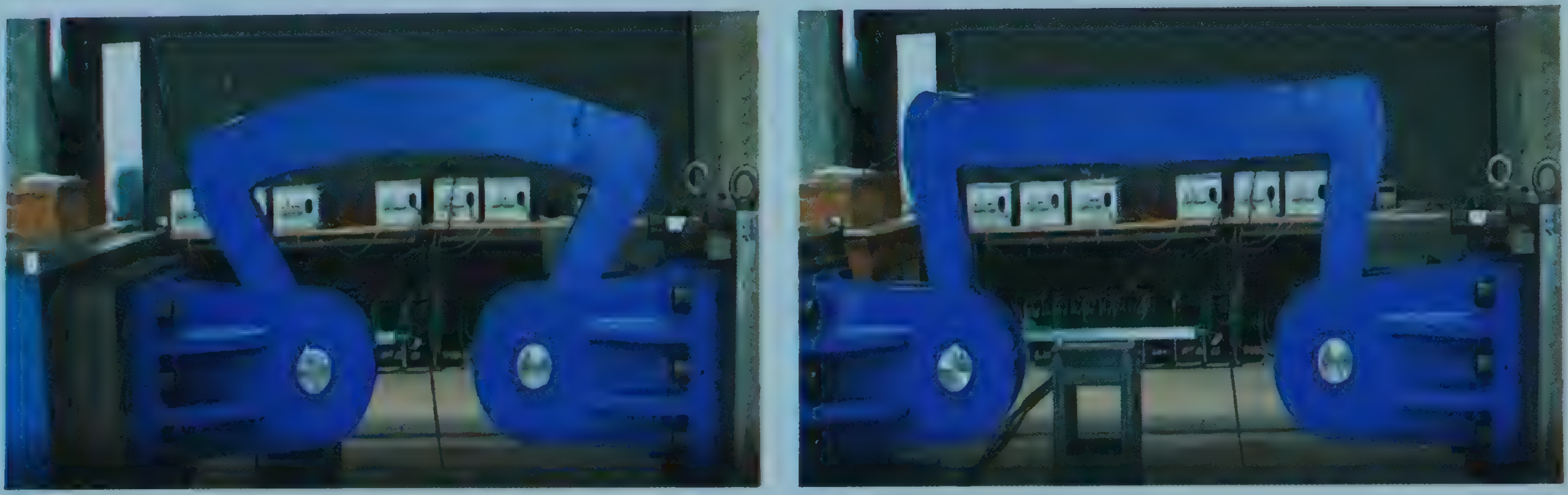


Fig. 11.30 Hysteretic Damper

11.4.2.5 Tuned Mass Dampers (TMD) (**Fig. 11.31**)

Cable Stayed Bridges are susceptible to issues relating to vibrations due to their low damping in combination with rather low natural frequencies. Vibration and vibration displacements are required to be damped which otherwise would render the structure non-usable under peak excitation or cause unacceptable discomfort. Also, resonance phenomenon can occur and structural collapse could happen. Fatigue phenomenon with crack in the structure can occur, which can lead finally to the structural collapse, too.

Tuned Mass Dampers are deployed to dampen the vibrations and uncontrolled displacements in the structure that are induced due to excitation forces from Wind, Seismic, etc.

As the name implies, a mass equivalent to the kinetic mass of the Structural component is suspended using springs and damping mechanism to act against the free structure swing or displacement thus controlling its movements.

TMDs offer following benefits:

i) Acceleration reduction for wind

ii) Low eigendamping → long decay durations

iii) Effective reduction of structural decay duration after the earthquake

A TMD is connected to the structure at the location where a significant or the biggest vibration is expected to occur. The TMD-device consists of a moving mass, springs and a damping element. The below sketches describe the principle of TMD and its main components. The TMD should be placed at the location of the greatest vibration amplitude in which case the efficiency is the highest with least effort.

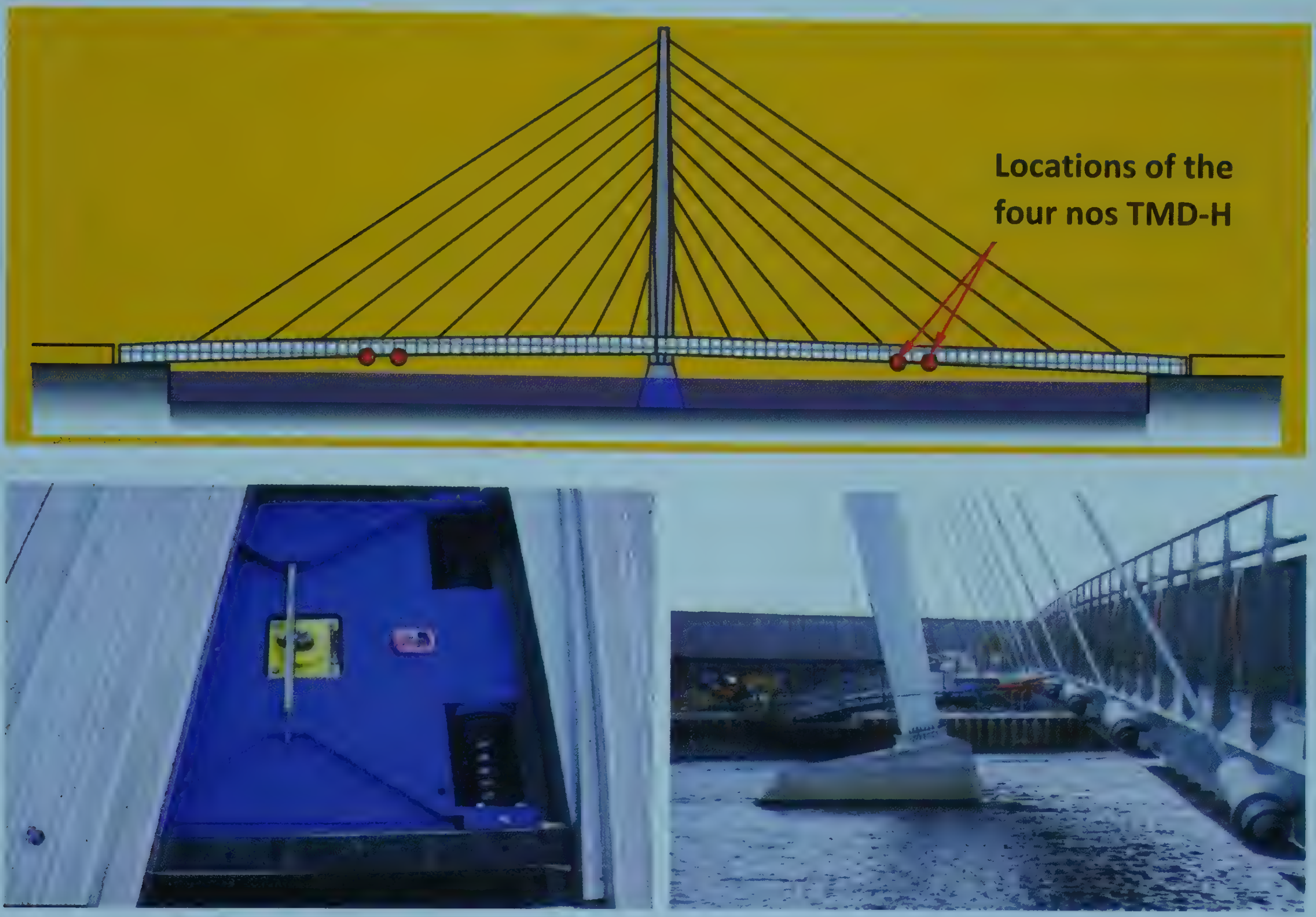


Fig. 11.31 Placement of Tuned Mass Damper (TMD)

11.4.2.6 Cable Dampers

Cables in a Cable Stay Bridge can vibrate in different modes due to varying excitation conditions:

To control multi-mode Cable Vibrations, passive, semi-active or real time adaptive Cable Dampers as discussed in detail under **Chapter 10** shall be used to dampen the varying vibration frequencies and to keep the Cables and Structure stable, saving from fatigue effects thus ensuring longer service life.

CHAPTER - 12

GEOMETRY CONTROL

Geometry control is an essential part of the construction monitoring process: appropriate profile and alignment are required as for any other bridge; in addition, geometry control is an essential tool to ensure that the stresses in the structure and stay cable forces meet design expectations.

12.1 Casting Curves

In all segmentally constructed bridges, one needs to cast the segments with camber corrections so that once all segments get erected /cast at site, one can achieve desired profile of bridge in its final shape.

Determination of pre-determined profile of segments such that the adjacent segments match each other, requires a detailed pre-camber control analysis, wherein all construction stages of erection of such prefabricated segments to build a complete bridge are modelled with accurate loading (temporary & permanent) activated at each construction stage (step) with most accurate predicted time line.

Time tag for each activity is very important so that time dependent effects of material used such as creep of concrete, shrinkage of concrete, relaxation effect of prestressing steel can be accurately accounted for. Such effects can cause deformation of each erected segment depending on stress level history in each segment during construction of bridge, age of concrete of segments at the time of its erection in bridge unit and age of concrete of segment at a construction stage at which deformations are required.

Each construction step analysis requires calculations of

a) Current step displacement ($\boldsymbol{\delta}_{ab}$) at each node:- It is displacement of nodes of selected construction stages due to activities performed within each construction stage (step)

b) Virtual Displacement at each node:- which occurs at a node before the corresponding node is even activated.

c) Accumulative / Net displacement nodes:- It is the net displacement of nodes up to end of the selected construction stage, i.e it is the summation of current displacement of node which occurs after the node is activated up to selected construction stage.

d) Real displacement of node:- It is the summation of virtual displacements which occurs before the corresponding node is even activated and net displacement, which occurs after the node is activated.

12.1.1 *Camber Control For Precast Segmentally Constructed Units*

A simple example of cantilever erection of three precast segments in three stages is given in **Fig. 12.1** Current step displacement and virtual displacement at each node at construction stages are shown. Such displacement values results are extracted by performing stage analysis for each stage separately under corresponding stage loading. Real displacement profile at the end of erection of each construction step is shown in last figure of **Fig.12.1**.

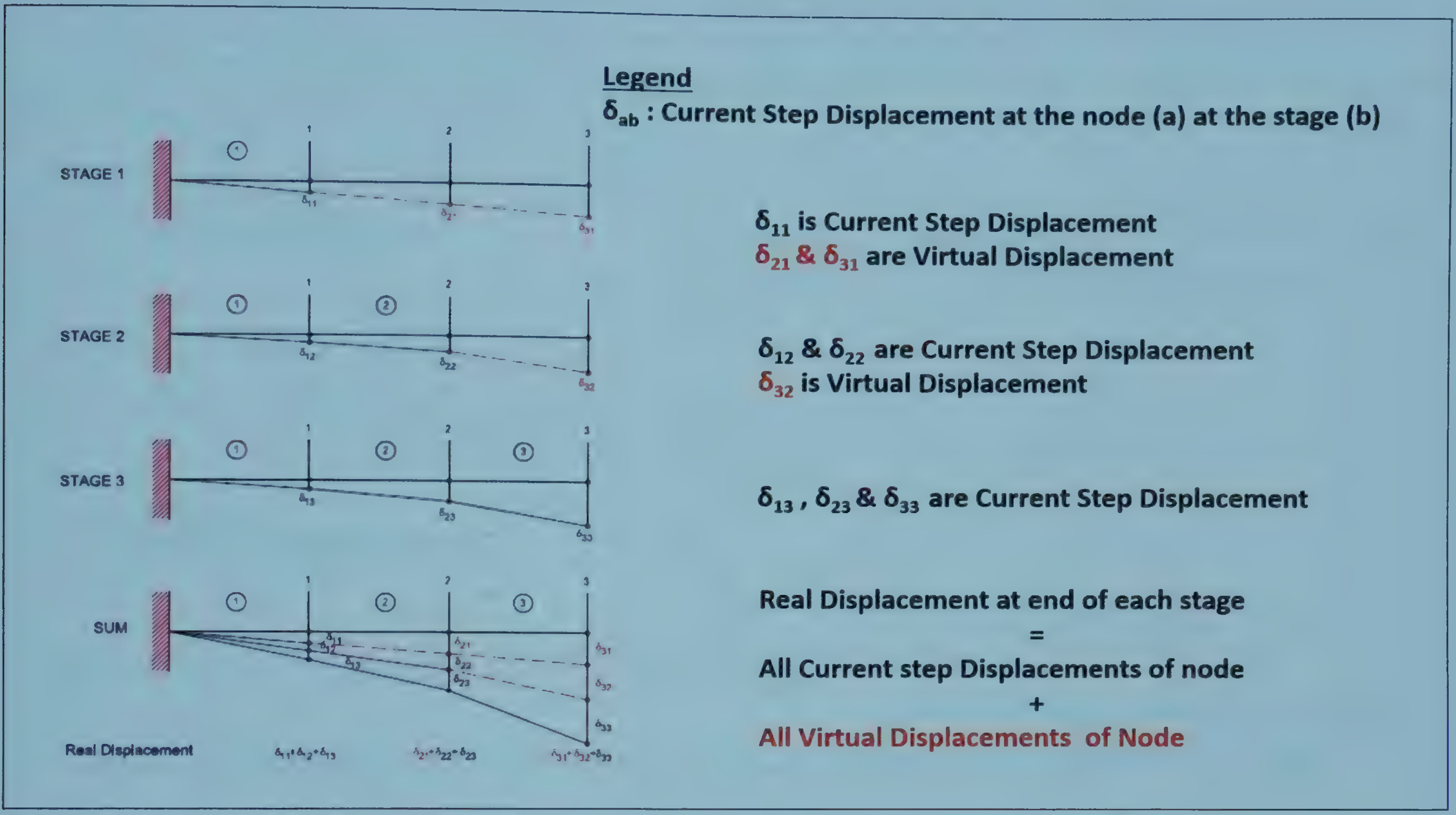

Legend
δ_{ab} : Current Step Displacement at the node (a) at the stage (b)
STAGE 1
STAGE 2
STAGE 3
SUM
Real Displacement
1
2
3
①
②
③
δ_{11} is Current Step Displacement
δ_{21} & δ_{31} are Virtual Displacement
δ_{12} & δ_{22} are Current Step Displacement
δ_{32} is Virtual Displacement
δ_{13} , δ_{23} & δ_{33} are Current Step Displacement
Real Displacement at end of each stage
=
All Current step Displacements of node
+
All Virtual Displacements of Node
$\delta_{11}+\delta_{12}+\delta_{13}$
$\delta_{21}+\delta_{22}+\delta_{23}$
$\delta_{31}+\delta_{32}+\delta_{33}$

Fig. 12.1 Real Displacement at Node (For Precast segments)

Profile of erected unit at end of each construction stage with, (i) no camber correction is done & (ii) camber correction done at each construction stage, is shown in **Fig. 12.2**

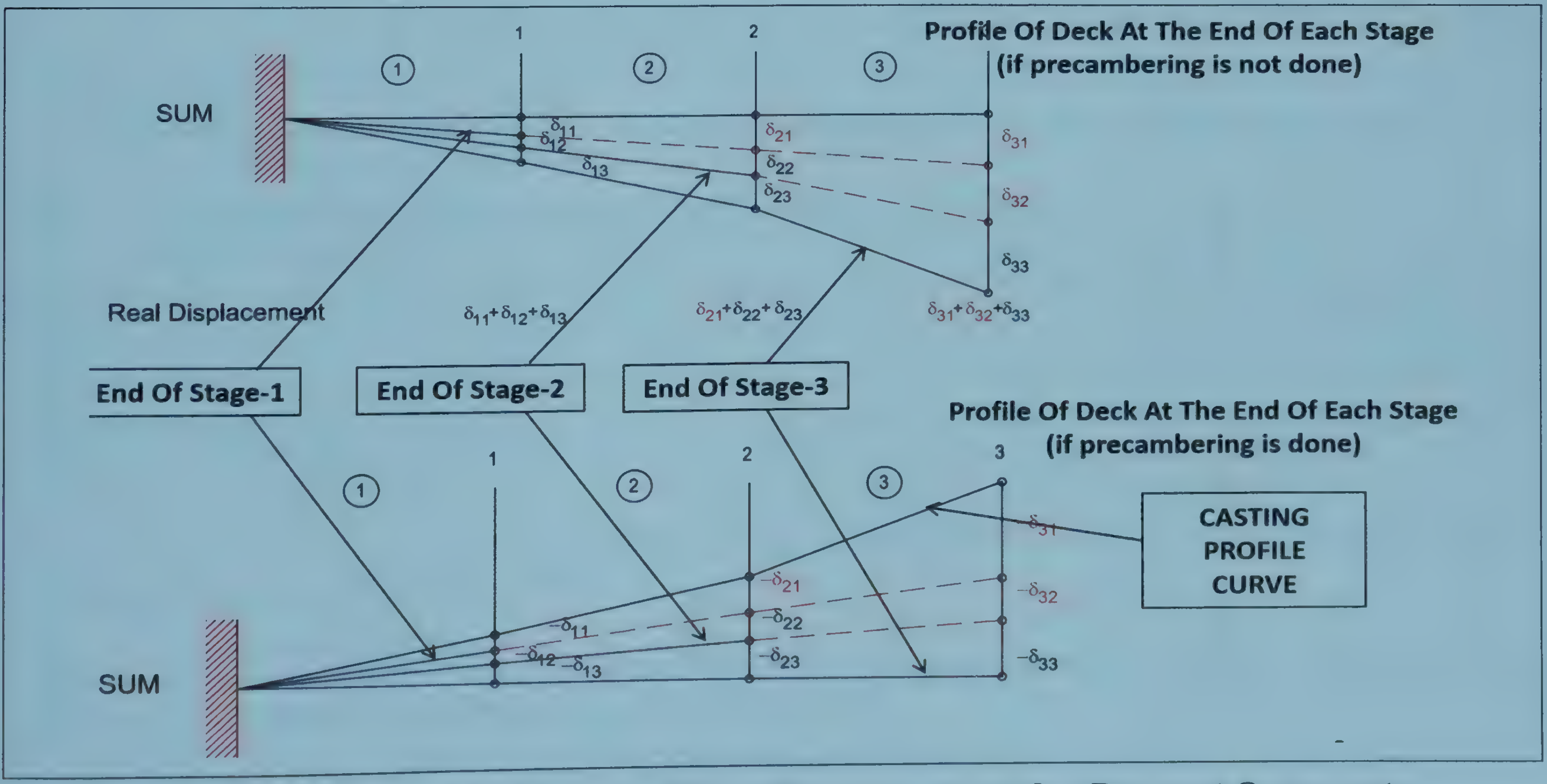

Profile Of Deck At The End Of Each Stage
(if precambering is not done)
SUM
1
2
3
①
②
③
δ_{11}
δ_{12}
δ_{13}
δ_{21}
δ_{22}
δ_{23}
δ_{31}
δ_{32}
δ_{33}
Real Displacement
$\delta_{11}+\delta_{12}+\delta_{13}$
$\delta_{21}+\delta_{22}+\delta_{23}$
$\delta_{31}+\delta_{32}+\delta_{33}$
End Of Stage-1
End Of Stage-2
End Of Stage-3
Profile Of Deck At The End Of Each Stage
(if precambering is done)
CASTING PROFILE CURVE
$-\delta_{11}$
$-\delta_{12}$
$-\delta_{13}$
$-\delta_{21}$
$-\delta_{22}$
$-\delta_{23}$
$-\delta_{31}$
$-\delta_{32}$
$-\delta_{33}$
SUM

Fig. 12.2 Precambering Using Real Displacement for Precast Segments

By reversing the final real displacement profile of erected unit at end of construction, one can work out casting profile curve of segments. The angle (ψ_i) between two segments to achieve a final structure with no deformation can be worked from real displacement data **Fig.12.3**. Such calculated camber correction of angle (ψ_i) between match cast segment is required to be provided while casting of segment.

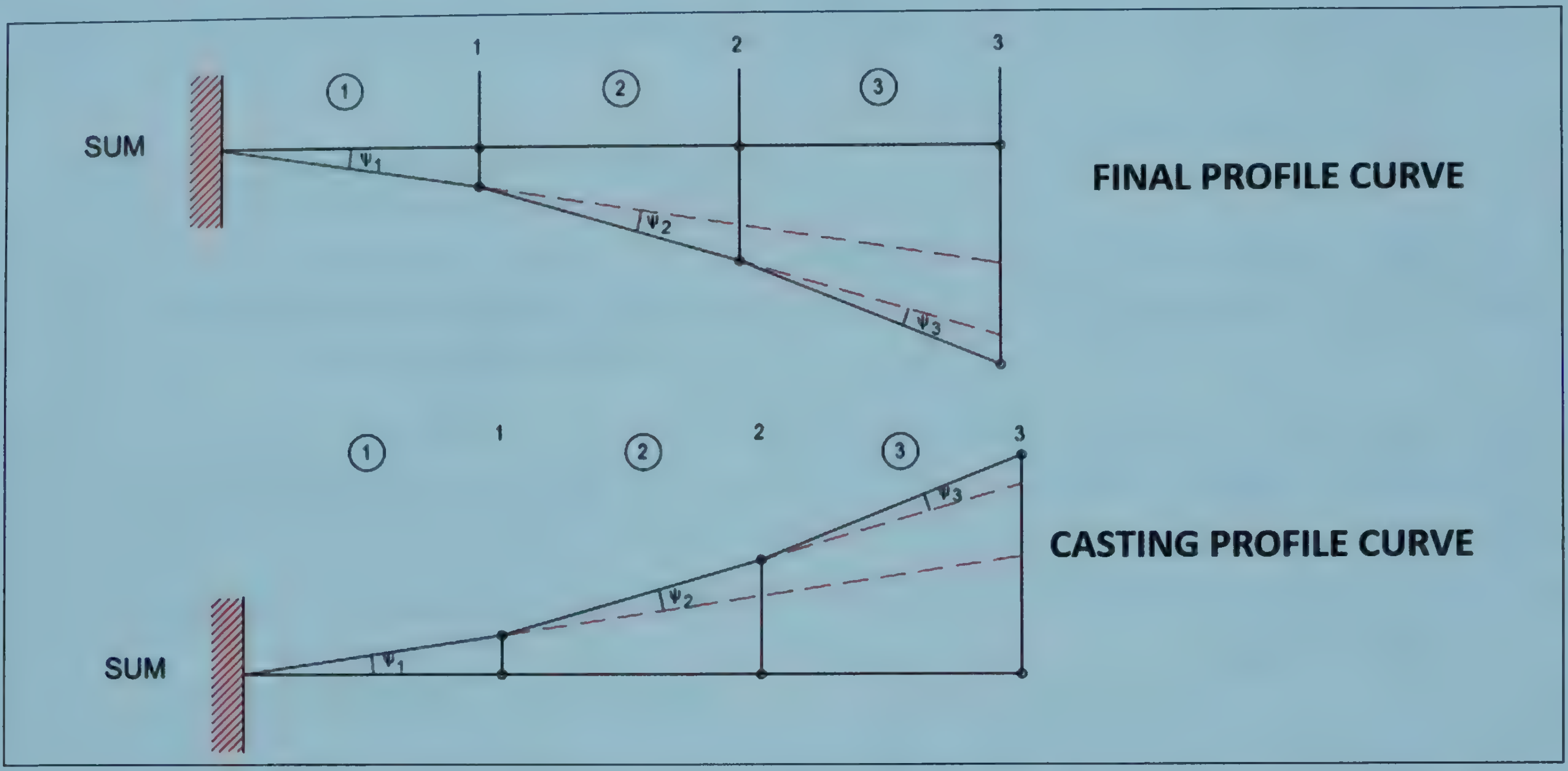

1
2
3
①
②
③
SUM
ψ_1
ψ_2
ψ_3
FINAL PROFILE CURVE
①
1
②
2
③
3
ψ_3
CASTING PROFILE CURVE
ψ_2
SUM
ψ_1

Fig. 12.3 Casting curve Using Real Displacement - For Precast segments

12.1.2 *Camber Control For Cast-In-Situ Segmentally Constructed Units*

A simple example of cantilever erection of three cast-in-situ segments in three stages is given in **Fig 12.4**. Current step displacement at each node at end of each construction stage are shown. Such displacement values results are extracted by performing stage analysis for each stage separately under corresponding stage loading.

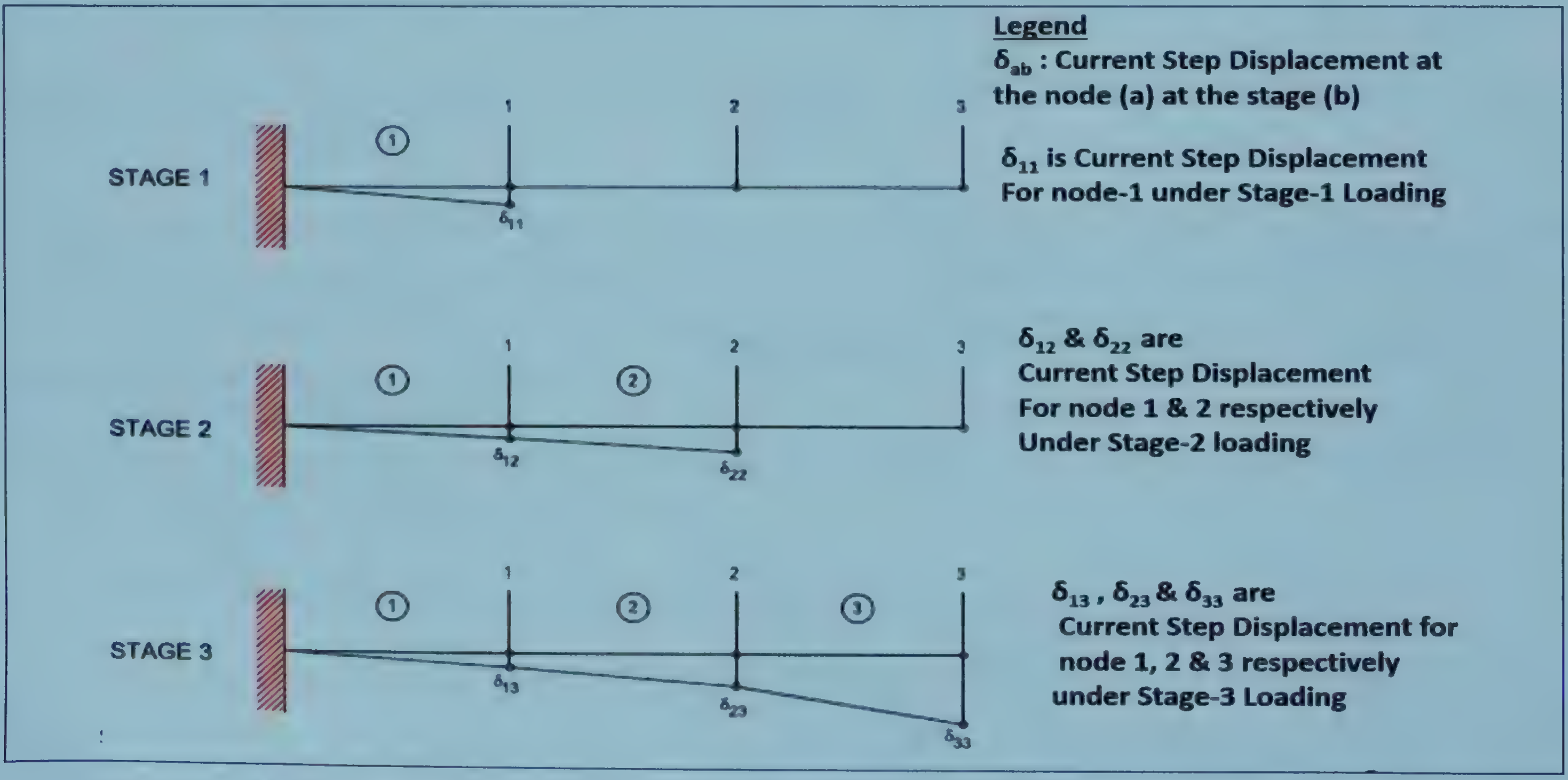

Legend
δ_{ab} : Current Step Displacement at the node (a) at the stage (b)
1
2
3
STAGE 1
①
δ_{11}
δ_{11} is Current Step Displacement For node-1 under Stage-1 Loading
STAGE 2
①
②
δ_{12}
δ_{22}
δ_{12} & δ_{22} are Current Step Displacement For node 1 & 2 respectively Under Stage-2 loading
STAGE 3
①
②
③
δ_{13}
δ_{23}
δ_{33}
δ_{13} , δ_{23} & δ_{33} are Current Step Displacement for node 1, 2 & 3 respectively under Stage-3 Loading

Fig. 12.4 Net/Accumulated Displacement at Node (For Cast-in-Situ Segments)

Net displacement profile at the end of erection of each construction step is shown in **Fig.12.5** (top figure) if no precamber is done.

By reversing the final net displacement profile of erected unit at end of construction, one can work out precamber value which needs to be provided at each newly casted node in each construction stage w.r.t final proposed profile curve of segments as shown in **Fig 12.5** (lower figure).

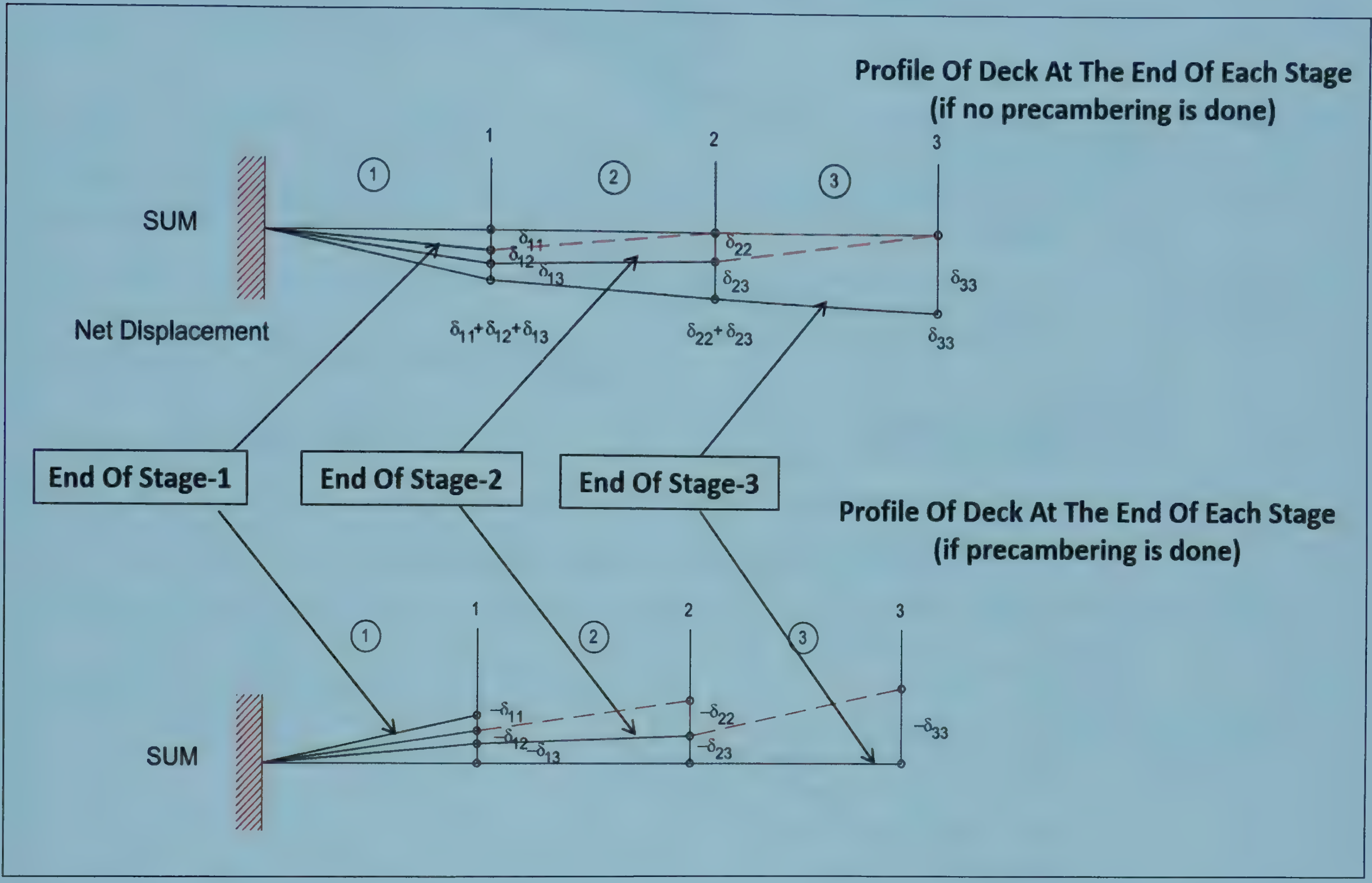

Profile Of Deck At The End Of Each Stage
(if no precambering is done)
1
2
3
①
②
③
SUM
δ_{11}
δ_{12}
δ_{13}
δ_{22}
δ_{23}
δ_{33}
Net Displacement
$\delta_{11}+\delta_{12}+\delta_{13}$
$\delta_{22}+\delta_{23}$
δ_{33}
End Of Stage-1
End Of Stage-2
End Of Stage-3
Profile Of Deck At The End Of Each Stage
(if precambering is done)
1
2
3
①
②
③
$-\delta_{11}$
$-\delta_{12}$
$-\delta_{13}$
$-\delta_{22}$
$-\delta_{23}$
$-\delta_{33}$
SUM

Fig. 12.5 Precambering Using Net Displacement For Cast-In-Situ Segments

12.2 Geometry Control for Precast Box Girder Segments

Segments are usually match-cast by the short cell method. Each segment is equipped with 4 survey markers (recommended at top of webs) for elevation control and 2 for alignment control (recommended at center line). The location of these markers is recorded in the casting cell reference system. "As-cast" plots are developed in this reference system **Fig. 12.6**.

The as-cast set of data is then transformed into the reference system of the erection site by a simple matrix operation **Fig. 12.7**.

It should be noted that a systematic error of only 0.3 mm on elevation measurements in the casting cell would result in significant discrepancies at the end of a long cantilever. The first segment of the cantilever must be placed accurately to avoid offsets at the cantilever end.

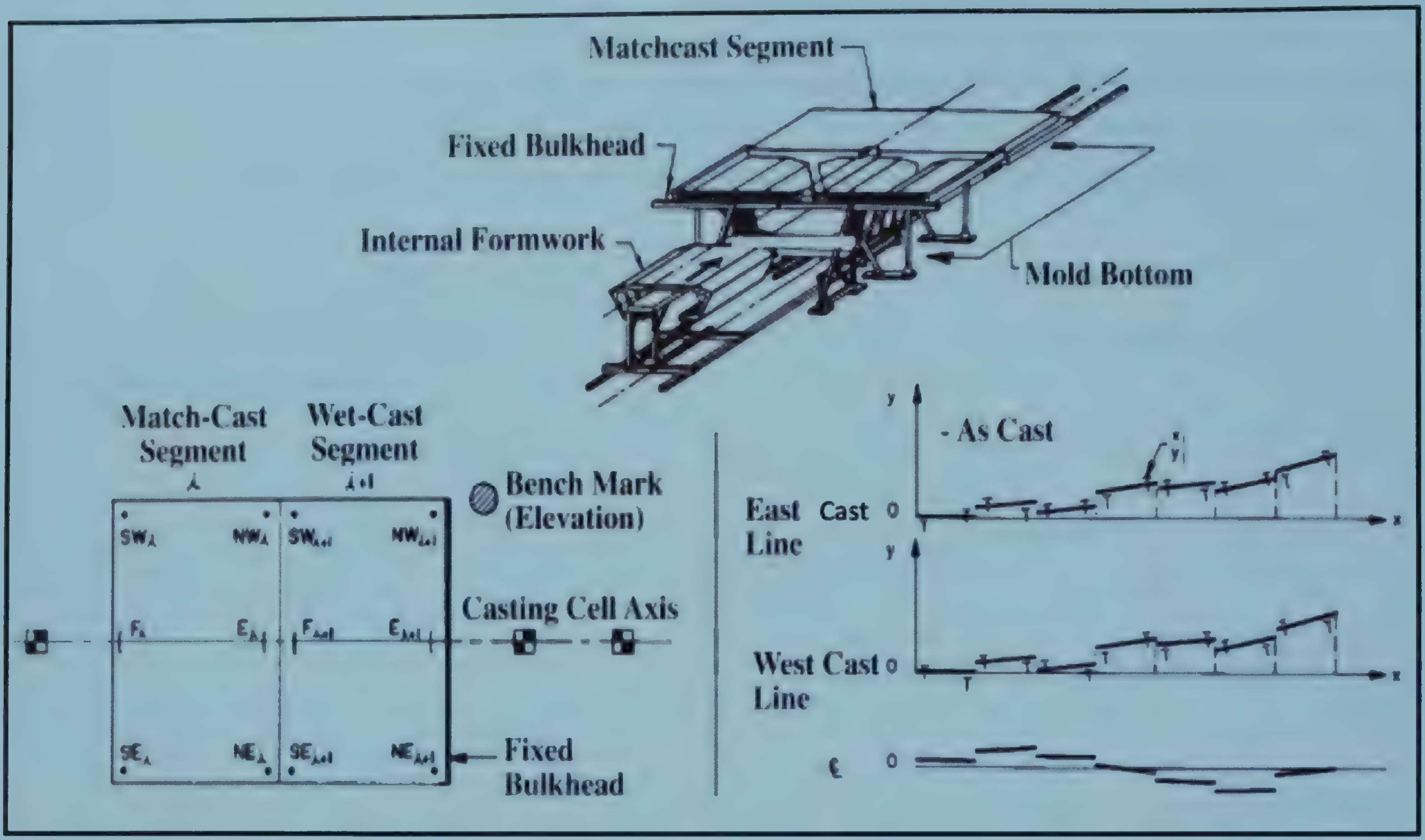

Matchcast Segment
Fixed Bulkhead
Internal Formwork
Mold Bottom
Match-Cast Segment
i
Wet-Cast Segment
i+1
Bench Mark (Elevation)
SW_i
NW_i
SW_i+1
NW_i+1
F_i
E_i
F_i+1
E_i+1
Casting Cell Axis
SE_i
NE_i
SE_i+1
NE_i+1
Fixed Bulkhead
- As Cast
y
x
x_i
y_i
East Cast Line
0
West Cast Line
℄

Fig. 12.6 Short Line Casting Method

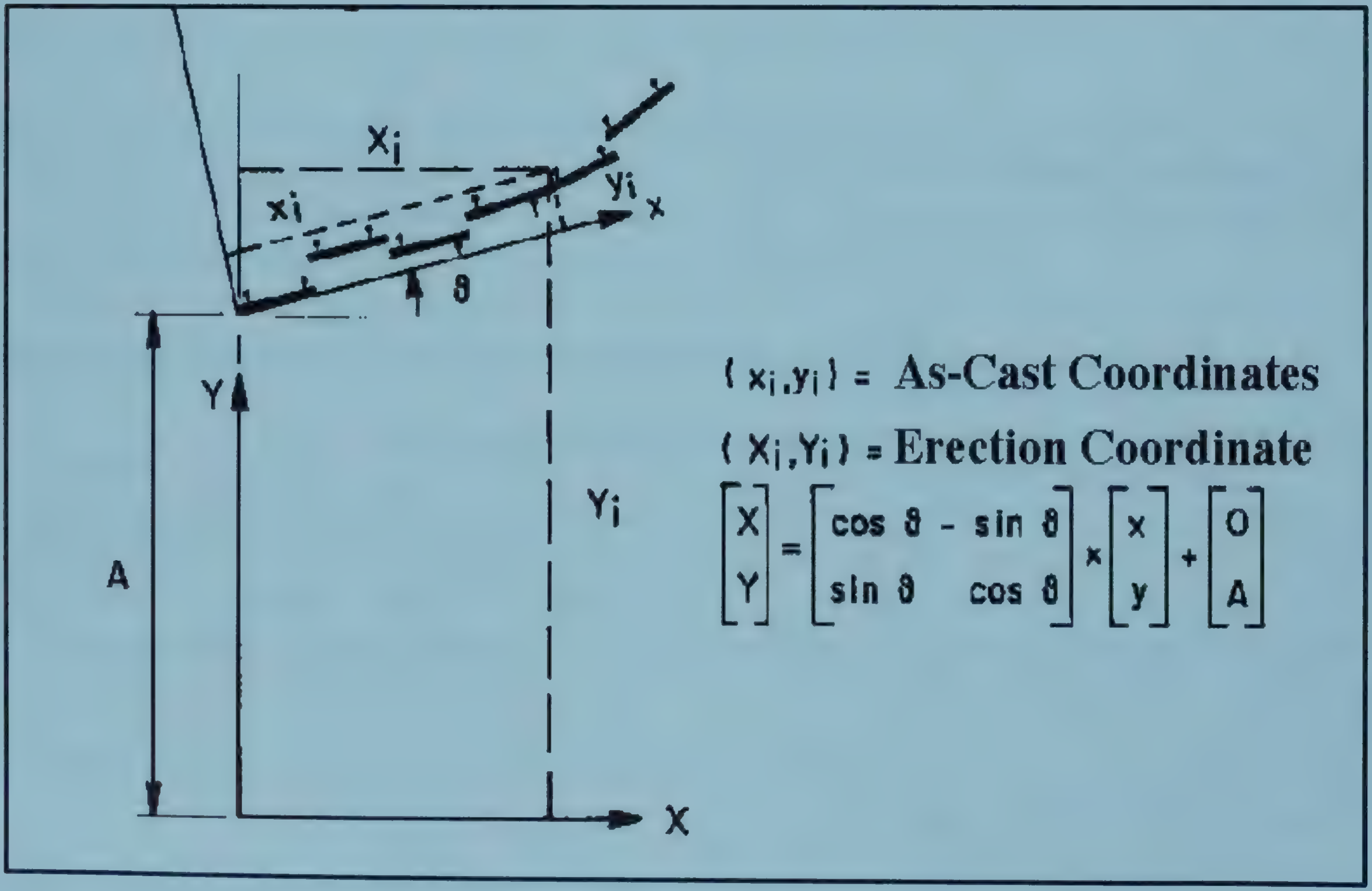

X_i
x_i
y_i
x
θ
Y
A
Y_i
X
{x_i, y_i} = As-Cast Coordinates
{X_i, Y_i} = Erection Coordinate
[X; Y] = [cos θ - sin θ; sin θ cos θ] × [x; y] + [0; A]

Fig. 12.7 Coordinate Transformation

Accuracy of controlling the horizontal alignment is more critical parameter especially when wider precast is planned and constructed. Wider deck such as more than 20 m width is commonly adopted in India. Misalignment of horizontal alignment result in more significant problem in closure joint stage as it's rectification in horizontal discrepancy is extremely difficult either by using the shimming plate or forced adjustment.

Deflections of the structure at each stage are obtained from the general time-dependent computer program. Adding these values to the as-cast survey marker elevations provides expected elevations at each stage of erection.

The elevation of the cantilever end should be plotted at each phase against theoretical **(Fig. 12.8)** Instant movements of deck and pylon should be measured when lifting a segment or stressing a stay cable and checked against theoretical figures. Interpretation of the results would give indications on the actual stiffness of the structure.

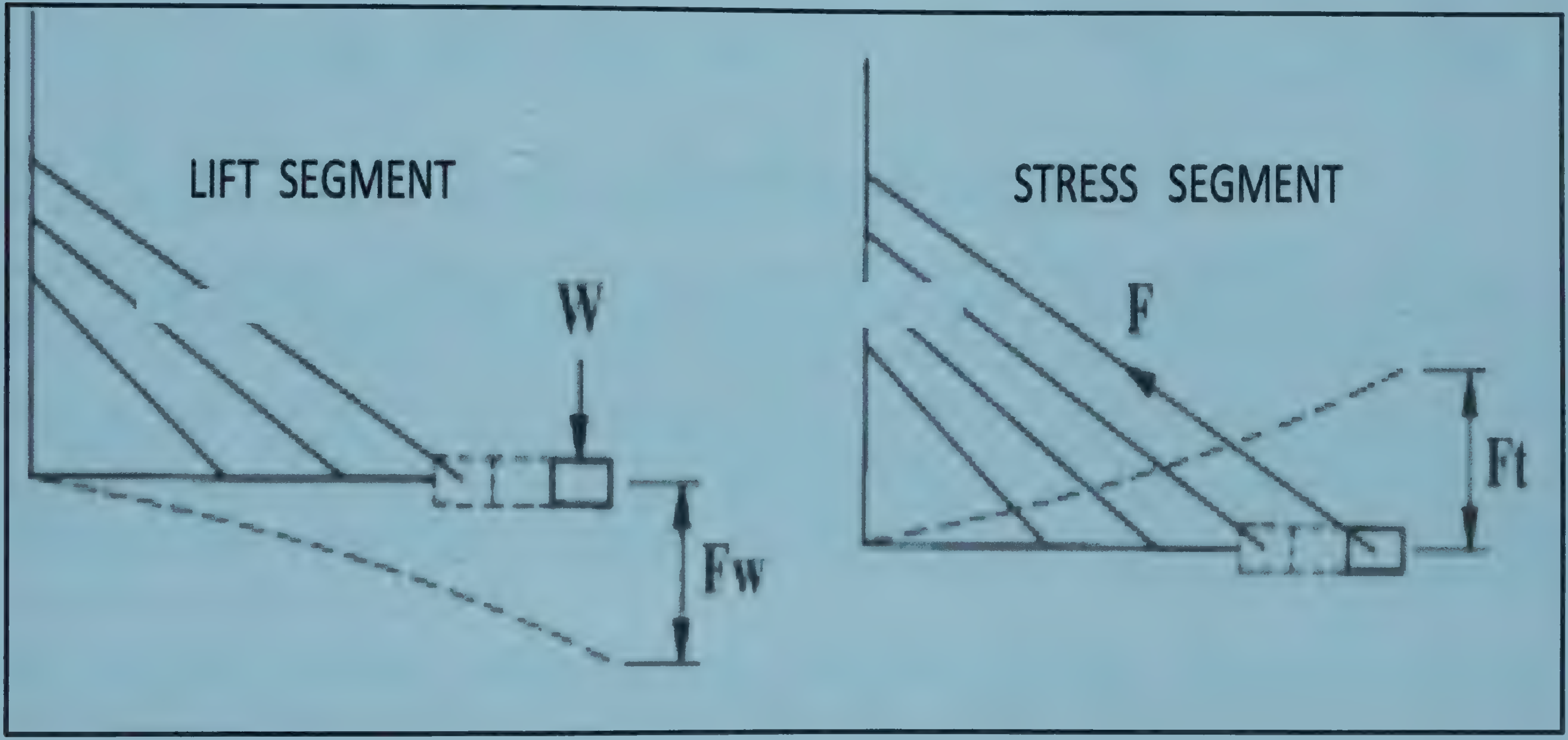


Fig. 12.8 Deformations During Construction

Bending of the overall cantilever can be checked by plotting actual and theoretical elevations at each joint at a given erection phase. After each deck survey, the verticality of the pylon should be checked. If the pylon is out of position, it means that the structure is subjected to unbalanced loads; and the deck survey would be affected by this condition.

It is imperative to check that the last erected segment is not warped. This can simply be done by comparing readings of the 4 elevation survey markers after erection and in the casting cell **(Fig. 12.9)**.

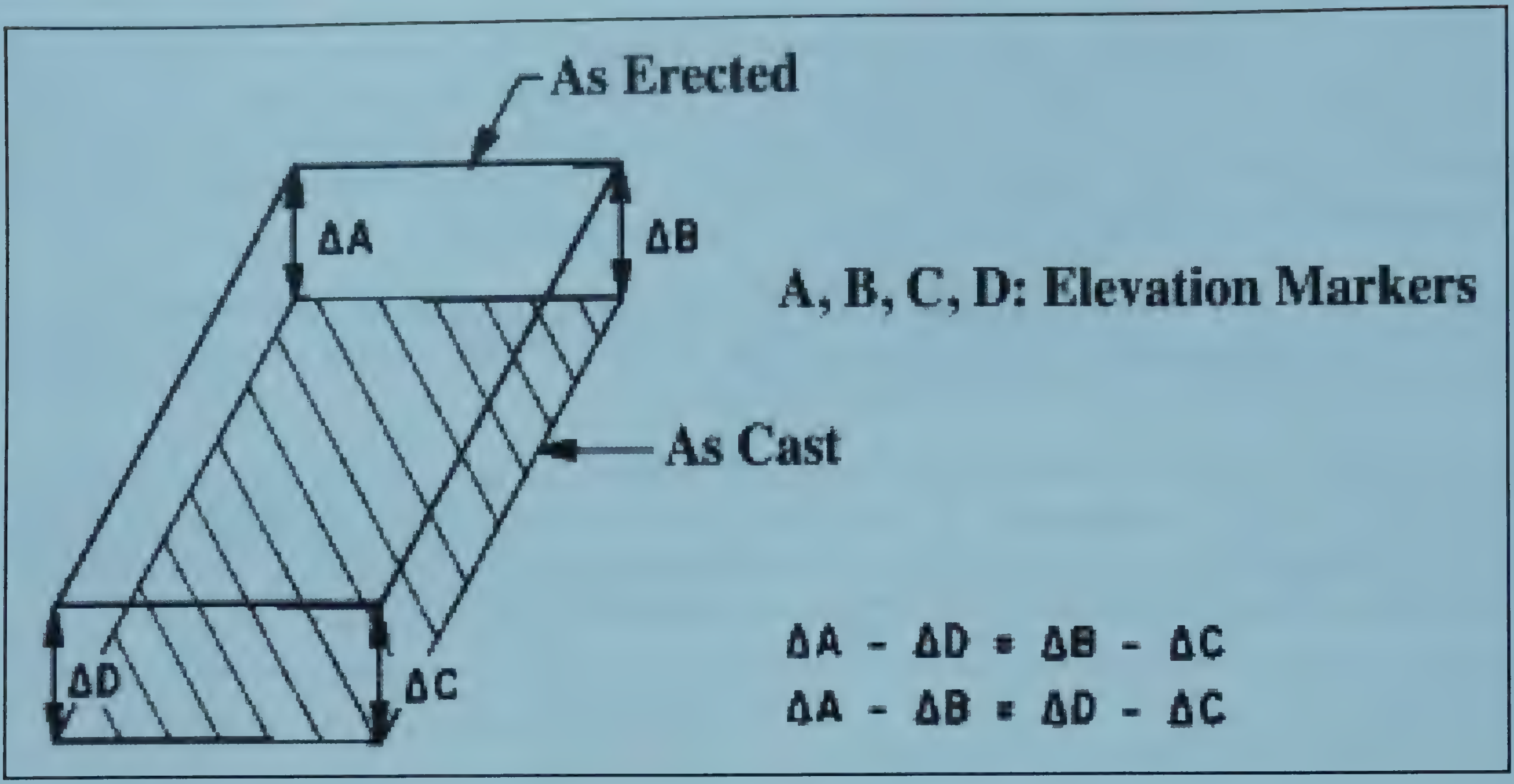


Fig. 12.9 Segment Shape Control

The accuracy of the stay cable jacking force can also be checked by measuring variations of elevations of the same survey marker during a full erection cycle (segments lifting, stay cable stressing and re-stressing). Comparing the result with theoretical figures allows determination of a gain or loss of elevation for the full cycle. Loss of elevation could indicate a deficit of effective stay cable force or excessive downward deformation, for instance, because of additional segment weight. Since the total deck movement includes an equal number of upward and downward deflections, the influence of the actual structure stiffness on the results is reduced.

The thermal gradients within the superstructure and between concrete deck and stay cables can affect stay cable forces. The nominal stay cable forces given in the design normally do not account for thermal effects. One solution consists of verifying and adjusting the stay cable forces early in the morning if the stay cable has to be stressed during the day-time in order to satisfy the contractor's operations. It is more practical to estimate the gradients at the time of stressing, for instance, with thermo-couples. At each stage of erection, a correction (**Fig.12.10**) of stay cable force can be computed to account for loss due to thermal gradients. During the daytime, thermal gradients create a downward deflection of the cantilever. When the gradient dissipates, the cantilever deflects back up and it results into a loss of stay cable force. Therefore, when stressing during daytime, an increase of stay cable jacking forces is required.

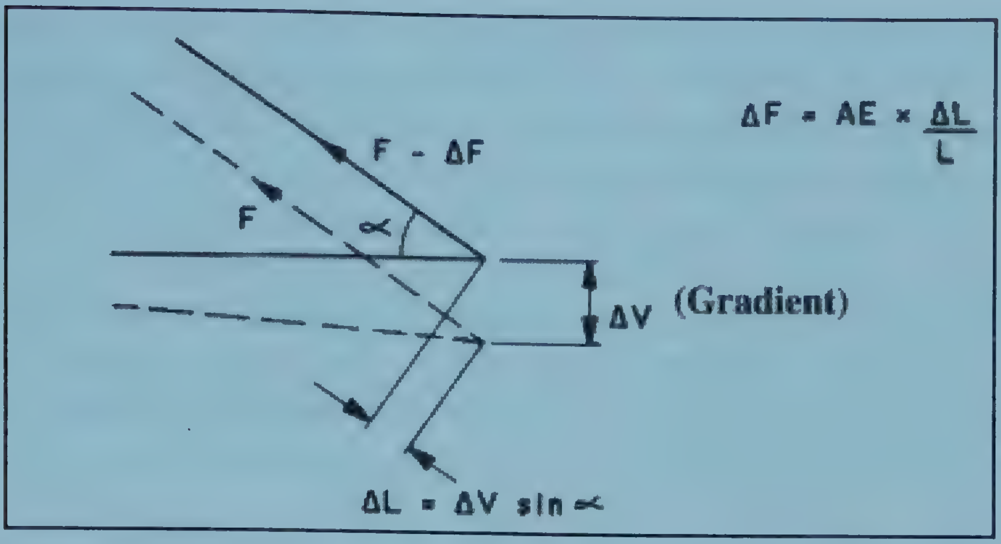


Fig. 12.10 Thermal Effect Stay Force Correction

Plotting of the cantilever end elevations **(Fig. 12.11)** at each phase gives a good indication of the actual stiffness of the structure.

The deck deflects up and down under stay cable stressing and segment lifting. The amplitude of the deflections increases with the length of the cantilever.

If the bridge is stiffer than assumed, the cantilever tip elevation will seem high after lifting a segment. After stressing a stay cable, the elevation of the cantilever will seem low.

Modifying the stay cable forces to bring the deck to the theoretical elevation at each phase could result into unacceptable stresses in the superstructure. This explains why a cable stayed bridge with a rigid type superstructure cannot be erected following theoretical elevations only. In this case the erection should be governed by stay cable forces with a tolerance not exceeding +/- 5%.

If cantilever tip elevations are low after stressing and lifting a segment, this is an indication that the whole cantilever is aimed low or is deflecting excessively.

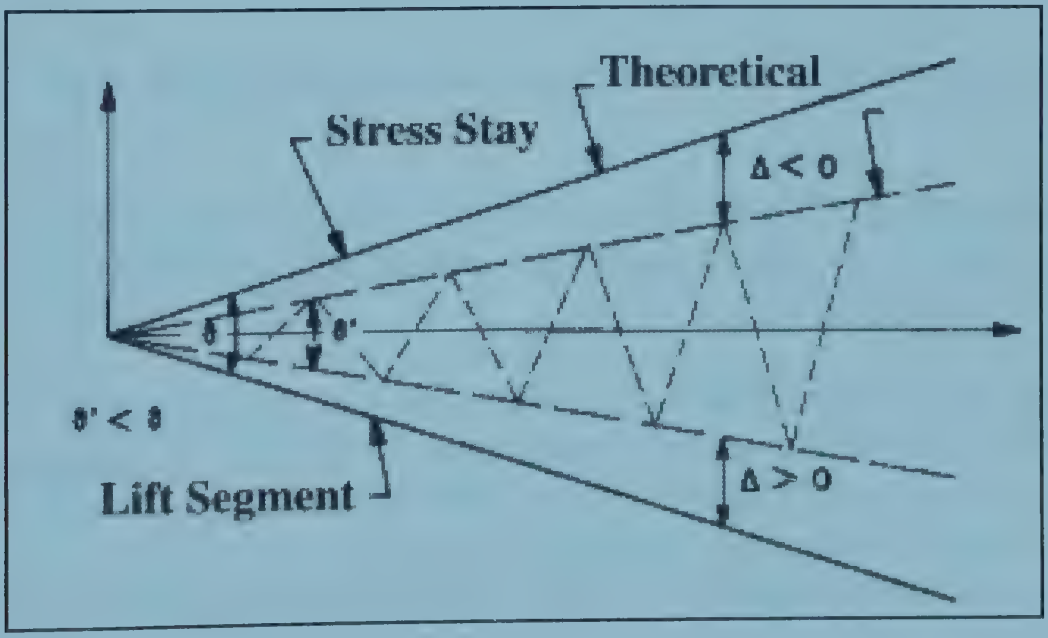


Fig. 12.11 Plot of Cantilever Tip

Shimming the joint to adjust the profile during the erection may be required similar to the precast balanced cantilever erection if the deviation of erected profile is large such that adjustment by stay cable force is insufficient to rectify. This case would happen more in case where girder is stiffer and span length comparably shorter. (i.e., stiffer deck compared with span length).

The American practice is to use the shims made of ASTM 240 type 304 wire cloth (roving). The GRP (Glass reinforced polymer) shim plates are recommended as the modulus of elasticity of hardened epoxy glue is closer to GRP avoiding potential stress concentration. In any case the maximum thickness of shims shall not exceed 6mm.

12.3 Geometry Control for Cast in Place Box Girder Segments

The geometry control procedures differ slightly from the procedures used for precast box girders: local geometry adjustments can be made with the traveling forms instead of shimming the joints between precast segments. Like with precast box girders, geometry discrepancies due to additional segment weight or inaccurate stay cable jacking forces need to be corrected by re-stressing the stay cables.

12.4 Geometry Control for Flexible Decks

Except for the initial casting stages close to the pylon, the deck elevations can vary greatly with a small variation of the stay cable force. At the same time, stresses in the deck are not affected significantly by variations of the deck geometry. It is therefore logical to prioritize the deck geometry control over stay cable jacking forces. Variations of segment weights are then automatically compensated for but it is important to record the stay cable forces to verify that allowable stay cable stresses are not exceeded. Again, it is advisable to provide some additional capacity in the stay cables at the design stage to account for these weight variations.

Since the erection procedure is guided by the deck geometry, it is essential to apply the stay cable jacking force at a time when the geometry of the structure is not affected by thermal gradients, that is early morning before sunrise.

12.5 Geometry Control for Steel-Concrete Composite Decks with Precast Concrete Slabs

Composite deck, consisting of longitudinal steel girders and floor beams onto which precast or cast-in-situ concrete is connected via shear connectors is advantageous for construction of bridge spans in the range of 400 m - 600 m.

For the construction of composite steel-concrete decks of cable-stayed bridges, methods of erection and analysis have to be applied that, upon completion of the deck, accurately yield the prescribed dead load configuration of the deck regarding geometry and forces. During deck erection, no unwanted bending moments should be locked into the composite section when the concrete slab is connected to steel girders and floor beams. The advantage of using precast concrete deck slab is to minimze the inclusion of locked-in bending and the respective effects of concrete creep & shrinkage.

The aim should be to eliminate or at least minimize the possible effects of concrete creep & shrinkage on the geometry and forces of the deck in order to simplify analysis of the deck erection, as well as to increase the accuracy of the geometric predictions. These effects can be minimized if the deck is erected so that the internal forces for the dead load configuration are those of the continuous beam on rigid supports. This is so because in a continuous beam on rigid supports, the sum of the positive and negative bending moments- that is, the span wise overall curvature- is zero. Moment variations due to creep eliminate each other (due to the axial forces in the deck, the deck slab can be assumed to be un cracked). The initial moment distribution does not change with time, and therefore time-dependent effects due to moment creep do not have to be considered. This method minimizes locked-in curvatures and the respective moments in composite decks. The typical erection cycle for a deck segment is defined by four stages and a two-step tensioning procedure for the cables as briefly explained below:

1) Erection of prefabricated welded steel grids consisting of longitudinal main girders with a cable anchorage at their tips and cross girders by the cranes and bolted to the deck.

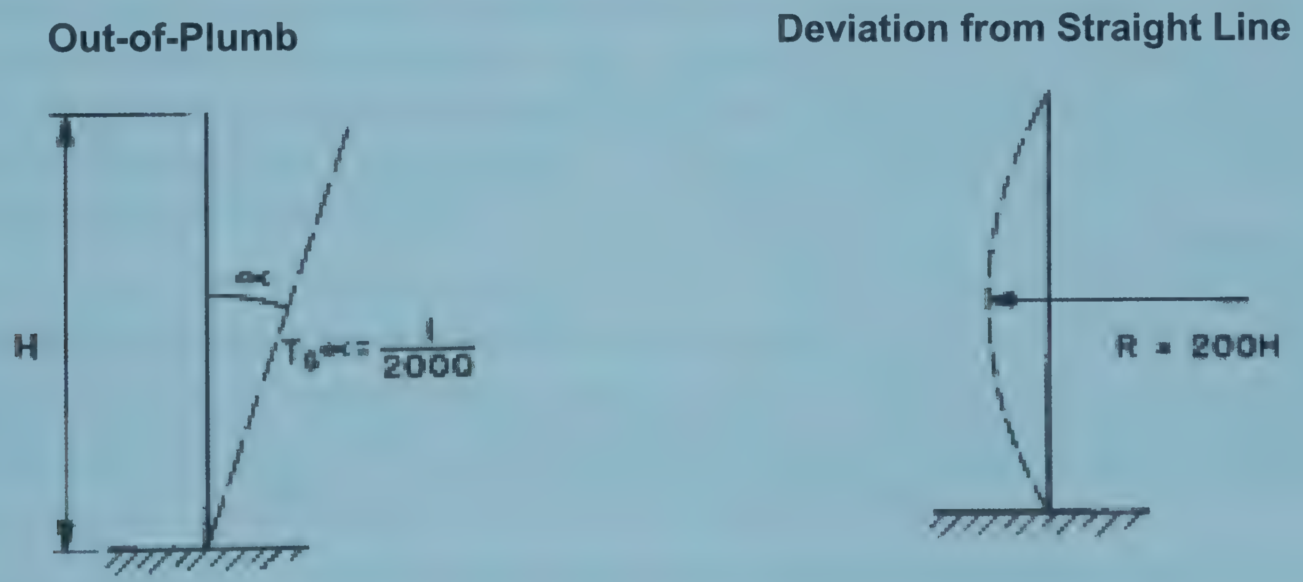
2) Installation and stressing of stay cables.

3) Erection of concrete decking panels and placement of in-situ concrete in closure joints between panels.

4) Re-stress the stays cables once the closure joint concrete has attained the required strength.

12.6 Geometry Control of Pylon

The geometry of the pylon **(Fig. 12.12)** must be controlled during construction to verify that the maximum out-of-plumb and out-of-straight-line values assumed in the design for this compression member are not exceeded.


Out-of-Plumb

Deviation from Straight Line

Fig 12.12: Pylon-Construction Tolerances

12.6.1 *Geometry Control of Steel Pylon*

12.6.1.1 Geometric Control during Fabrication

Invariably steel pylons are constructed by prefabricated segmental construction method where segments are fabricated in an established fabrication workshop and then transported over to site, while they would be reassembled before erection.

Some of the major challenges involved in fabrication of steel pylon are :

- Use of plate having thickness ranging from 25 mm even up to 250 mm including Z quality steel. Adoption of appropriate welding sequence to avoid deformation
- Preheating up to temperatures in access of 600 degree for butt welding of thicker plates
- Stress relieving arrangements due to heavy welding
- High precession and tolerance requirement for bolt holes and end milling
- Drilling millions of holes for HSFG bolts ranging from 12 to 36 mm in very thick plates.
- Availability of Skilled Manpower
- Requirement of special lifting equipment's during fabrication and trial assembly
- Availability of well-equipped fabrication facility.
- Machining facility to carry out sometimes even up to 36 m long surface at one go.

Adopting the most appropriate welding procedure is very important to avoid distortions and deformations for welding of pylon segments, and to ensure high level of accuracy and quality. Fabrication accuracy would directly affect geometrical dimension of the whole segment after their trial assembly, which were to be precisely matched within 1/10 of mm. Number of NDT testing like Dye penetration test, ultrasonic test, etc. have to be carried to ensure the desired quality of the product.

All the compression joints are normally designed to transfer load through bearing contact surfaces along with splice connection. To achieve appropriate bearing surfaces, all the contact surfaces in compression like main girder edges, pylon horizontal joints have to be machined to have a surface planes not more than 0.3 mm per meter.

Following are some typical steps recommended in the fabrication process :

- Technical groundwork – preparing fabrication drawings, workout requirement of material, equipment's and support arrangements.
- Procuring and testing of Raw material

- Cutting of material as per approved fabrication drawings.
- Panel welding and NDT testing
- Fit up of an segment and NDT testing
- Initial survey and dimensional check for fit-up segment. Conduct rectifications if required
- Machining end surface of the segment in contact with other surface at splice, which transfers load through bearings.
- Match drilling
- Trial assembly, match drilling and final survey
- Blasting and painting.

Trial assembly at fabrication yard is an important prerequisite for prefabricated segmental erection of steel pylon similar to that match casting in concrete segmental construction. Trial assembly has to be performed in line with the sequence of erection at the fabrication factory / bridge site. The trial assembly jig has to have enough rigidity to avoid subsidence.

The special attention regarding the following key points are warranted to decide the fabrication, trial assembly and geometric control procedures :

- The accurate geometric dimension of pylon structure with edge considering their complex configuration if any with heavy welding in volume, the control in welding shrinkage and in the structure deformation during and after welding.
- The quality of thick-plate welding.
- The requirement on the accuracy of bearing contact in end face considering their huge vertical compression of the pylon tower and its shaped alignment.
- The trial assembly has to be carried out to ensure the geometrical alignment. The jig platform should be set in a rigid and accurate way that its features would not affect the accuracy in the trial assembly and final adjustment of sectional segments, consisting of complex configuration of leg if any, main body with front and rear cable stay structures.
- Keep the temperature changing in an acceptable range to constantly maintain the precision in fabrication of the pylon.
- Due to the huge size of the pylon segment with heavy weight, to avoid any deformation or prevent any damage done to the machined contact surface of the segment and the structure itself, during transportation.
- Geometrical alignment of pylon segment together with the site welding control.
- Control in painting quality.

12.6.1.2 Geometric Control during Erection

Cable stayed bridges are usually highly unstable structure during erection. This demands a well-coordinated and calculated step by step erection procedure including rigging method statements for pylon erection.

The main design issues to be addressed in order to erect the pylon are:

- How to stabilize the pylon at its base during construction;
- How to recover structure deformation during erection;
- How to install large, heavy box girder segments that had to be placed, one above the other, in a position especially when there are double inclination;
- How to manage the temperature effect, together with the tight tolerance of a bolted structure.

A detailed stage-by-stage analysis has to be done to determine stress distributions in the structure for all intermediate stages and to perform structural adequacy checks for erection stage. Sufficient strengthening of permanent structure has to be provided based on construction stage analysis wherever required.

A careful monitoring of the geometry throughout construction is required to ensure that the structure behaved as predicted and the target geometry is met. Since many of the quantities involved in the construction are only approximate values (exact elements dimensions and weight) or can be measured with a given tolerance (stay cable lengths and forces) and environmental conditions change during construction, and the calculation model itself is only an approximation of the real structure, a continuous monitoring of structural deformation during the construction is of fundamental importance.

Temperature changes, mainly during the day due to direct sunlight, modifies the real/actual geometry of the structure, continuously changing the position of working points making it difficult to install the structural elements in their theoretical position. The movements also occur due to stressing and construction effects in addition to environmental effects as above.

Survey data has to be collected and examined for all the relevant erection stages, based on survey points which are marked during trial assembly. The survey points at deck and pylons have to be established with the aim of identifying the movements independent from installation of new elements and cable stressing and separate environmental actions effects (temperature, sun's radiation, wind) from construction effects.

CHAPTER - 13

INSPECTION, MAINTENANCE AND STRUCTURAL HEALTH MONITORING

13.1 Introduction

Performance of cable stayed bridges during design service life shall be achieved by implementing maintenance management plan drafted during the design stage.

The Main Tension Elements (MTE) within a cable bundle are, in most cases, hidden from the view of inspectors. Access to cables for visual inspections or non-destructive testing (NDT) is generally difficult, and in the case of the anchorage zones, nearly impossible. Those who are tasked with inspection and maintenance of stay cables are faced with challenges for which proven and accepted methodologies and tools are limited and, in many cases, very costly.

Though IRC:SP:35 and IRC:SP:18 can be referred for inspection and maintenance of substructure, foundation and other auxiliary components, this Chapter focuses on deck, stay cable, and tower inspection, maintenance and SHM of cable stayed bridges. This includes the overall frame work of SHM, design of SHM system, the parameters that can be monitored and the corresponding sensors, data management and interpretation of results. SHM may be used to verify the design assumptions, to evaluate the performance of the bridge during its service, especially under extreme events, and when exposed to various environmental conditions, and, for maintenance management.

Annexure A5 gives guidelines for the Organisation Setup and Training Requirements for Inspection and Maintenance.

13.2 Inspection

Regardless of quality of design and construction it is inevitable that a bridge structure will suffer some degradation due to the effects of age, environment and loading that could often exceed the specified design values. The extent of this degradation is also influenced by the extent of the inspection and maintenance programme. The design life of long span bridges is generally achieved by specifying durable components, detailing for easy inspection access, having regular inspection and maintenance performed, and replacing components that have reached their service lives. Therefore, inspection access shall be an integral part of design.

Adequate space should be provided to allow access for construction, inspection and maintenance personnel. The space should also allow for material and equipment to be delivered from the deck or ground level to the cable anchors for repair and replacement work. Generally, appropriately sized access openings and interior lighting or a power source should be provided. The design should provide sufficient points of access and minimum opening sizes for inspection and maintenance.

13.2.1 *Scheduling of Inspections*

Different type of inspections and frequency as given in **Table 13.1** may be used if not incorporated in the operation and maintenance manual. **Table 13.2** gives the inspection requirement component-wise.

Table 13.1 Scheduling of Inspection

Types of Inspection	**Inspection goals**	**Frequency**
Initial inspections	First inspection performed and inventorized before the bridge is first opened to traffic to identify whether there is a change or update in inspection procedure for future maintenance.	One time
Routine Inspections	Visual inspection of the components given in **Table 13.2**.	Initially, the periodicity can be once in a month for six months. Subsequently the duration between two inspections can be increased to 6 months, depending on the condition of bridge component.
Detailed inspections	This should be a visual inspection from a close distance, NDTE can be used to identify cause of distress (see **Table 13.2**)	Once in three years/ or when major distress observed during the routine inspection.
Special Inspection/ Emergency Inspections	Inspections carried out immediately after earthquake, cyclone, flood, fire, impact damage etc. Can be done by foot with naked eye or with binoculars and with the help of appropriate NDTE (See **Table 13.2**)	After natural /manmade disasters such as earthquake, flood, cyclone, fire etc

Table 13.2: Type of Inspections and Parameters ~~to be Checked~~

Bridge component	**What to inspect/measure**	**Inspection type**				
		Regular		**Special/Extraordinary After**		
		Routine	**Detailed**	**EQ**	**cyclone**	**other**
Deck	Deflection/level	▲	√	▲	▲	▲
	crack	√	√	√	√	√
	vibration	▲	√	▲	▲	▲
Tower	crack	√	√	√	√	√
	tilt/inclination	▲	√	√	▲	√
Stay cables	vibration	▲	√	▲	√	▲
	tension	▲		▲	▲	▲
Cable pipe	Damage Deformation Discolouration	√	√	√	√	▲
Cable Anchorage	Damage, deformation, corrosion	√	√	√	√	√
Vibration control devices	Damage Deformation Rust or deterioration Leakage of viscous material	√	√	√	√	▲

√ Perform the inspection, ▲ Perform inspection if needed

13.2.2 *Inspection Program*

A typical inspection program of cable stayed bridges includes but is not limited to the following:

i) Change in structural geometry

ii) Deflections due to dead load and live loads

iii) Inspection for unusual cracks or deformations especially in concrete components subjected to high stress

iv) Distress/damage of expansion joints and uneven road level on either side of joint

v) Evidence of corrosion to reinforcement, other embedded steel and prestress / stay cable

vi) Erosion and abrasion of the substructure due to fast moving particles / objects in the river

vii) Collision damage

viii) Fire damage

ix) Deterioration of concrete due to chemical agents

x) Integrity of corrosion protection

xi) Scouring of foundation

The design reports along with "As Built Drawings" and construction records, form the basic data, for maintenance during operation stage. Therefore, it is essential to store these valuable data for a long period.

It is very much necessary to measure and record the initial inclination of tower and cable tension during the initial inspection of bridge, these form the basis to identify the abnormality of these components. It would be appropriate to perform the maintenance rationally and efficiently by creating a database of various records, to make it possible, to refer updated records of repair, inspection and monitoring at any time.

These guidelines will be used for inspection and maintenance of all components of cable stayed bridge superstructure. For the stay cable system, for recording the findings during inspection, the checklist given in **Annexure A6** shall be used.

13.3 Maintenance Aspects

13.3.1 *Maintenance issues to be considered during design*

Maintenance issues required to be addressed during the design stage are as follows-

i) Identify the design details that can give rise to structural and durability problems,

ii) Design details that can cause difficulties for inspection

iii) Provisions of access for inspection and (i) Inspection traveller(s) /underslung traveller/ maintenance carrier if required in case mobile bridge inspection cannot be used for bridge inspection.

13.3.2 *Maintenance Manual*

Contract documents shall specify the need for a maintenance manual. A maintenance manual shall be seen as an integral part of the design, as regular inspection and maintenance is required to achieve the recommended minimum design life of 100 years

The designer of a cable stayed bridge shall prepare a manual providing information about the basic demands on the components used in the bridge, the sequence and methodology of construction of the bridge, and suggestions about the inspection procedures of the bridge.

This Maintenance Manual shall be prepared after the completion of construction. Based on the design and construction, key areas for inspection need to be identified and the exact period between inspections for each component shall be included in the Manual. The Maintenance manual shall be approved by the client.

Special components such as the friction pendulum bearings and modular expansion joints will have a shorter cycle time than the main bridge structure due to their susceptibility to damage.

The maintenance manual shall contain at least the following:

i) Reference to the Design Documents (inclusive of the Corrosion Protection Plan)
ii) Reference to the As-Built Records and any inaccuracies that could not be rectified
iii) Provide the specific time table and procedures for inspection, testing, maintaining and record keeping for the non-replaceable bridge components
iv) Provide the specific time table and procedures for inspection, testing, maintaining, replacing and record keeping for the replaceable bridge components
v) Indicate the qualification of inspection and maintenance team required for each activity
vi) Provide operational instructions and specifications of all mechanical and electrical equipment, if any

13.3.3 *Minimum Maintenance Requirements*

i) The maintenance team needs to have a full set of the drawings and maintenance schedules that indicate the frequency of inspection and maintenance for each part of the structure.

ii) The maintenance plans should also highlight areas where something has gone wrong during construction and which need special monitoring See also (13.3.2) (ii)

iii) For maintenance, repair, rehabilitation and strengthening of various concrete components of cable stayed bridge IRC:SP:40 shall be referred.

iv) The geometry of the cable stayed bridge shall be surveyed at regular intervals, and particularly after a major event, such as, a strong wind storm, an earthquake, deep scour due a major flood etc. Anomalies in the geometry, if identified in a reliable survey, tend to reveal distress or defects which shall be thoroughly investigated in order that remedial actions can be formulated and taken.

(iv) Where scour poses a significant risk to the structure, a scour monitoring plan should be developed as a part of the bridge inspection and maintenance program. Inspection requirements, including method and frequencies, should be specified.

(iv) For maintenance, repair and replacement of stay cable system, guidance from stay cable manufacturer/supplier shall be sought.

13.3.4 *Bridge Management System*

A comprehensive scientific bridge management system consists various tasks related to collection of data for inventory, to period of inspection, details of inspection of each component and procedures to monitor, analyze and implement the activities related to maintenance works. Authorities can use the bridge management system to achieve optimum use of available funds, adaptation of bridges to traffic needs and public safety.

Data Base Module

A pre-requisite for attempting any scientific system of management of bridge maintenance is to have a proper data base module comprising inventory, inspection records and maintenance, repair and rehabilitation data.

Inventory

The inventory module shall consist of records of (i) administrative data (ii) general data (iii) technical data (iv) geometric data (v) hydraulic data (vi) geotechnical data (vii) environmental data (viii) structural design data (ix) structural drawings and (x) photographs of bridge and various components before the bridge is open to traffic.

Bridge inspection record

Bridge inspection record consists of data gathered and recorded while checking the condition of various spans and following up the deterioration process from one inspection to another. This could include (i) initial inspection report, (ii) routine inspection report, (iii) detailed inspection report, and (iv) special inspection and or emergency inspection report.

Bridge maintenance, repair and rehabilitation data file (MR&R)

This module is the collection of all the data pertaining to the details of each component of Bridge/Viaduct, its inspection, its condition etc. In addition to the basic design data and construction data, it contains maintenance, repairs or improvement work data including as-build drawings, major repairs, and exceptional events such as earthquakes, abnormal floods etc. The data as stored provides adequate information to make decisions for repair & strengthening works.

13.4 Structural Health Monitoring (SHM)

13.4.1 *Brief of Existing Guidelines for SHM*

There are many existing guidelines for SHM of structures in different countries like Canada, UK, European Union, China, Australia, Switzerland and USA developed during the last two decades. Since the first structural health monitoring (SHM) guide, Guidelines for SHM **[1]**, was released, new SHM standards have been added around the world. The Canadian intelligent sensing for innovative structures, which published SHM of Innovative Canadian

Civil Engineering Structures, introduced basic techniques in SHM **[2]**. It also covers topics including repairing and strengthening of structures (use of fiber-reinforced polymers), intelligent sensing (integrated fiber optic sensors, fiber bragg grating sensors), economic significance (service life of structures will be extended with SHM), remote monitoring, and intelligent processing **[2]**.

The European Union developed the structural assessment, monitoring, and control (SAMCO) association for the maintenance of infrastructure and disaster prevention **[3]**. The SAMCO database is mainly used for infrastructure maintenance, and especially for bridge management. The SHM in the UK is applied to different civil infrastructures (e.g., dams, bridges, offshore installations, nuclear installations, etc.). In the UK, dams were the first structures for which legislation mandated SHM inspections, though recently more attention has focused on bridges **[5]**. Recommendations for SHM implementation in the UK include low-cost system (investment) and high benefit (return). The implementation of sensors, i.e., the total quantity of sensors that will be put on a structure, their positioning, the distance between them, and the algorithms and processing should be appropriately determined beforehand to avoid data overloads, including number of measurements, technical requirements, and different levels of accuracy for different sets of information collected. Environmental effects, including noise, on SHM systems could seriously influence or inhibit detection. In conclusion, the development of SHM in Europe provides good ideas and instructions for future SHM standards in other countries.

The rapid emergence of new standards and codes related to civil engineering in China has also led to the development of new SHM regulations **[4]**. More specifically, in 2016 the Chinese Ministry of Transportation published the first governmental SHM code regulating the installation of smart sensors in bridges **[6]**. China's ability to develop and implement new civil engineering designs, while rapidly increasing the size and volume of structures, can serve as a model for transportation agencies in other countries in writing, approving, and implementing their own new SHM regulations **[7]**.

Although the SHM standards from these institutions might not be enforced by their governments, the standards still give regulatory instructions for the implementation of SHM, which is supportive toward SHM code development.

13.4.2 *Need for SHM*

Structural health monitoring is a non-destructive in-situ structural sensing and evaluation method that uses a variety of sensors attached to, or embedded in, a structure to monitor the structural response, to analyze the structural characteristics for the purpose of estimating the severity of damage/deterioration and evaluating the consequences thereof on the structure in terms of response, capacity, and service-life **[2,8,9].**

Some key issues for cable stayed bridges that warrant the consideration of health monitoring systems include:

- Towers and cable elements of long-span bridges make the structure susceptible to the uncertainties of wind and its aerodynamic effects.
- Complex or proprietary devices used on long-span bridges often require monitoring for optimal performance. They are often used to mitigate between competing

demands on flexible long-span structures, i.e. the need to make the structural system rigid for dynamic effects such as wind, and flexible for others, such as seismic effects and differential settlements. High performance devices, such as lock-up devices, isolation bearings, tuned mass dampers, etc., are often needed to mitigate extraneous construction costs.

- Present design trend includes consideration of the high cost of total bridge replacement, and the trend to extend design life through the use of durable high-performance materials in conjunction with replaceable/sacrificial elements.
- Currently, the cost of inspection and maintenance of long-span bridges is relatively high.
- Consideration should be given to the nonlinearity and variability of parameters associated with the design of long-span bridges

SHM can be classified as short-term and long-term monitoring and the situations in which they are adopted is given below.

Short-term monitoring consists of live-load and dynamic testing. Data is only collected over a relatively short period of time when these field tests are taking place. The collected measurements can provide information about structural performance such as load and moment distributions, stress/strain levels, and serviceability issues. In addition, the collected data will provide valuable insights for future designs, and improve qualitative assessments of the bridge structures with different load tests.

Long-term monitoring usually consists of collecting ambient vibration and environmental data to evaluate structural condition over long periods of time. For instance, with long-term monitoring, changes or abnormalities in structural behavior can be detected without human interactions, or environmental effects (corrosion monitoring, seismic effects) on the performance of the structure can be quantified **[10,11].**

13.4.3 *Objective*

The main objectives of structural health monitoring include any/or all of the following:

i) design verification (short-term monitoring)

ii) traffic management (long-term monitoring)

iii) inspection and structural maintenance (long-term monitoring)

SHM is generally used to verify the design assumptions, to evaluate the performance of the bridge during construction as well as service, especially under extreme events, and when exposed to various environmental and loading conditions. This may be used for the maintenance and /or traffic management of bridges.

Where design life is predicated on minimum condition and performance of specific bridge components, such components shall be considered for instrumentation during the design stage.

13.4.4 *Overall Design Requirements including Institutional Set Up*

The complete design process of a SHMS can be summarized by following these steps:

1) Identify the need for monitoring
2) Set the objectives of the monitoring system
3) Undertake risk, uncertainty and opportunity analysis
4) Identify parameters and response requirements
5) Select appropriate sensors
6) Select sensor location and communication system
7) Install system and calibrate
8) Acquire data, process, manage and report
9) Assess data/study of report
10) Decide on the need for remedial action

The design process covers several traditional study fields, such as civil engineering, electrical engineering and computer science. Therefore, the Institute or the organisation undertaking the design and implementation of SHM should have bridge experts, electrical, electronics and communication and computer engineers. They should have trained staff for installation and calibration of sensors. They should have proven track record in design, installation and performance monitoring of at least two cable stayed bridges.

Parameters, Location and Number of Spans to be Monitored

While designing a SHM system, first step is to decide whether short-term or long-term monitoring is required as discussed in **Section 13.4.2**.

The structural response parameters to be monitored include strain, acceleration and displacement of deck at critical locations, cable forces, strain and displacement of tower tip.

The environmental parameters include wind intensity and direction, water level, atmospheric temperature and humidity.

There are other loadings/ effect of actions to be monitored such as vehicular loading (traffic volume, type of vehicle, speed of vehicle, axle loading of trucks, axle spacing, distance between trucks) pedestrian loading/ crowd loading, thermal effects, seismic effects and wind effects.

Selection of location of sensors shall be done based on the three-dimensional analysis of cable stayed bridge under various loadings. It may be appropriate to identify the critical locations for performance monitoring to be identified by the structural designers and included in the structural drawing.

The minimum number of spans to be instrumented and monitored shall be one unit (between two expansion joints) if the purpose of SHM is to verify the design assumptions and to collect a baseline performance data.

All cables stayed bridges with main span length more than 300 m shall be selected for performance monitoring.

In the case of multi-span cable stayed bridge, with identical spans, at least one cable stayed unit shall be considered for SHM.

Cable stayed bridges provided with seismic isolation or vibration control devices shall be instrumented to monitor the performance of bridge

For more details on design and functional requirements of SHM, SHM frame work and safety evaluation of bridge using the data collected, **Annexure A7**.

13.5 Case Studies

This section gives examples of a few bridges which have been instrumented for Structural Health Monitoring purposes. These examples are given so as to provide typical arrangements of instrumentation. It is expected that when any particular bridge is considered for being instrumented, the exercise given in (**Section. 13.4.4)** will be carefully carried out in order to optimise the arrangement for its economy and effectiveness.

13.5.1 *Signature Bridge in Delhi [12]*

Bridge Details

The Signature Bridge is an eight-lane with semi-harp shaped cable-stayed road bridge across the Yamuna River, Delhi. It has an overall length of 575 m, and a main span of 251 m. The composite deck contains three main girders, with cross girders at 4.5 m spacing. The polygonal shaped tower provides, to a substantial extent, the stress balance required to support the deck. It also contains lifts and stairs providing access to the top of the 151-meter tower, for maintenance and sightseeing purposes.

Parameters to be Monitored and Location Selection

The following data are recorded by the system and the sensor layout is shown in **Fig. 13.1** and summarized in **Table 13.3**

Environmental Factors

Environmental conditions at the top of the tower are recorded by a weather station which measures the ambient temperature, relative humidity, precipitation levels, barometric pressure, as well as wind speed and direction. Wind speed and direction is measured by an ultrasonic sensor at the deck level. Relative humidity and temperature inside box section of the deck structure is measured by four hygrometers. The level of corrosion in the tower base structure is monitored by means of embedded chloride ion and corrosion sensors. In addition, seismic events, which are expected to have a serious impact on the entire structure, would be measured at the tower base in each leg plinth using 3D high-definition accelerometers.

Load Factors

Natural vibrations are measured by eight 3D-accelerometers, which will be capable of relocation along the length of the structure to provide total coverage on a cyclical basis. Traffic data is recorded by laser analysers, enabling lane usage, average lane speed and vehicle classification are to be determined. Traffic congestion, accidents and other anomalies will be captured by a video surveillance unit featuring colour cameras, capable of recording 30 frames per second and night-time usage.

Bridge Structural Response

Strain in the tower, compensated for temperature, is measured by welded strain gauges, while strain in the deck girders, also temperature compensated, is measured by bolted strain gauges. Stay cable forces is measured by electromagnetic sensors, weatherproof and telecommunications shielded, on the longest, shortest and medium length cables of each set of cables – two sets on the main span and one on the back span. Cable vibrations are recorded by 3D-accelerometers, which is detachable to allow relocation during and after the construction phase to assist in calculation and verification of damping, as well as assessment of changes in harmonics during operation. Tilt of the deck on each span and of the pylon (above and below the elbow) is measured by inclinometers, and displacement of the deck by strain gauges at the expansion joints.

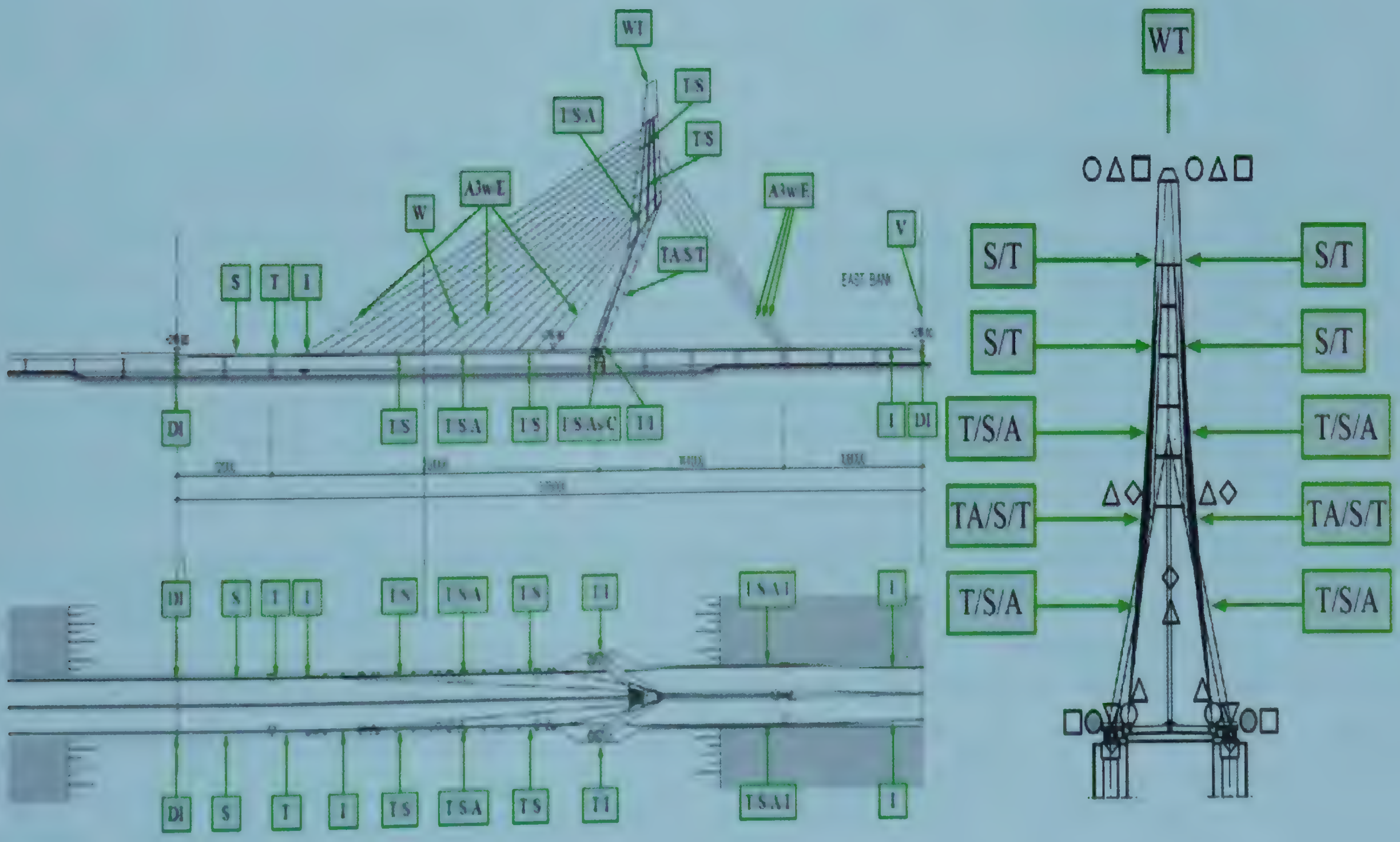


Fig. 13.1 Sensor Layout of Signature Bridge

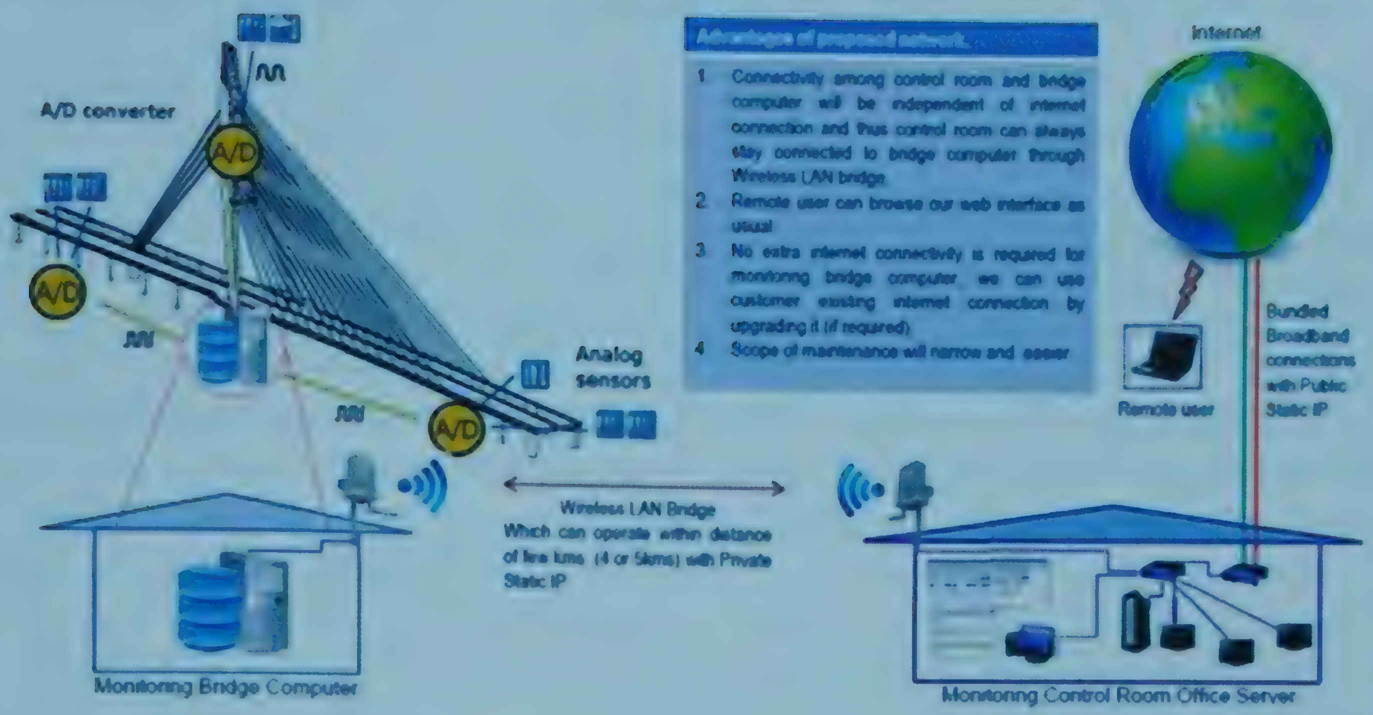
Data Acquisition, Transmission, Processing and Management

The data acquisition unit is based on industrial graded cPCI systems with dual-core CPUs, running data acquisition and data calculating software. The unit controls all sensor functions and provides all measured data with source codes (name and position of sensor) and time stamps to allow accurate time synchronisation. A/D-conversion is done by multi-channel, 24-bit converter modules, located as closely to the sensors as possible. All raw data measured by the sensors is stored on two independent hard discs to prevent data loss in case of disc failure. Pre-processing of the monitoring data also takes place here. The master station is equipped with independent time sources (GPS) to ensure self-sufficiency should connection to the local network be lost. This ensures preservation of records during critical events such as traffic accidents or earthquakes. The unit is also protected by an air-conditioned cabin to enhance reliability and durability.

Table 13.3 Summary of sensors included in the SHM system

Sensor	Logo	Sensors according to Specifications	Channels according to Sensors
		Sensors	Channels
Structural temperature	T	20	24
Strain gauge boltable	S	10	10
Strain gauge weldable/ glueable	S	10	10
Wind sensor 3D	W	1	4
Wind multi-sensor (Wind 2D direction and speed, ambient temperature, precipitation levels, barometric pressure, relative humidity)	WT	1	6
3D accelerometers, cables	A3w	18	54
3D accelerometer, deck & pylon	A	8	24
Seismic accelerometers	As	4	12
Displacement sensor	DI	4	4
Inclination, tilt	I	4	8
Corrosion	C	3	6
DV Camera	V	4	
Traffic analyzer	TA	8	
Electromagnetic sensors	E	9	9
Sum		104	171

The data connection between the master station on the bridge and the bridge operator's control room is by means of a wireless LAN bridge using a private static IP as described in **Fig.13.2.** This can operate over a distance of four to five kilometres. This system enables real-time data transmission, independent of mobile telecommunication service providers.


A/D converter
A/D
A/D
A/D
Analog sensors
Internet
Bundled Broadband connections with Public Static IP
Remote user
Wireless LAN Bridge
Which can operate within distance of few kms (4 or 5kms) with Private Static IP
Monitoring Bridge Computer
Monitoring Control Room Office Server

Advantages of proposed network

1. Connectivity among control room and bridge computer will be independent of internet connection and thus control room can always stay connected to bridge computer through Wireless LAN bridge
2. Remote user can browse our web interface as usual
3. No extra internet connectivity is required for monitoring bridge computer, we can use customer existing internet connection by upgrading it (if required)
4. Scope of maintenance will narrow and easier

Fig. 13.2 Data transmission

Evaluation of Structural Performance

Data presentation is controlled by a local monitoring data server in an office of the client's choice. Since the server is connected to the internet and the data transfer from the master station on the bridge to the server is wireless, the location can be freely chosen. All data received from the master station is stored permanently on two independent hard discs to prevent data loss in case of disc failure. The server is also capable of processing, managing and presenting data from any other permanent monitoring system that might be connected in the future.

The user interface enables an authorised user to log on to the system, and view any data which may be of interest, for any period of time during which measurements were made. It also enables the user to input condition and event values and, if necessary, instructions for personnel. Information is presented in a user-friendly and simply understandable format. High frequency data is displayed in clear graphics (e.g. mimic panel format). The user can also instruct the system to provide an alarm if predefined threshold values of any variable are exceeded. Such alarms can take the form of a visual notification on a monitor, or an email or SMS to a selected person. This user interface is accessible at any time, from anywhere in the world with an internet connection.

13.5.2 SHM description of a Steel Cable Stayed Bridge, Jindo Bridges, South Korea [13]

Bridge Details

The Jindo Bridges, (**Fig 13.3**), are twin cable-stayed bridges. Each of these bridges consists of three continuous spans, with a 344 - m central main span and two 70 - m side spans. The width of the first bridge is 11.7 - m with the design traffic velocity of 60 km/hr and the width of the second bridge is 12.55 m with traffic design velocity of 70 km/hr. It has a streamlined steel box girder supported by 60 high-strength steel cables connected to two towers.

Parameters to be Monitored and Location Selection

The first Jindo Bridge has 38 strain gages, four inclinometers, two anemometers, two seismic accelerometers, five uniaxial capacitive accelerometers, and 15 uniaxial piezoelectric accelerometers. The second Jindo Bridge has 15 thermometers, 15 strain gages, four biaxial inclinometers, two string pots, two laser displacement meters, 24 Fiber Bragg Grating sensors, 20 uniaxial capacitive accelerometers, two biaxial force balance type accelerometers, and three triaxial seismic accelerometers.

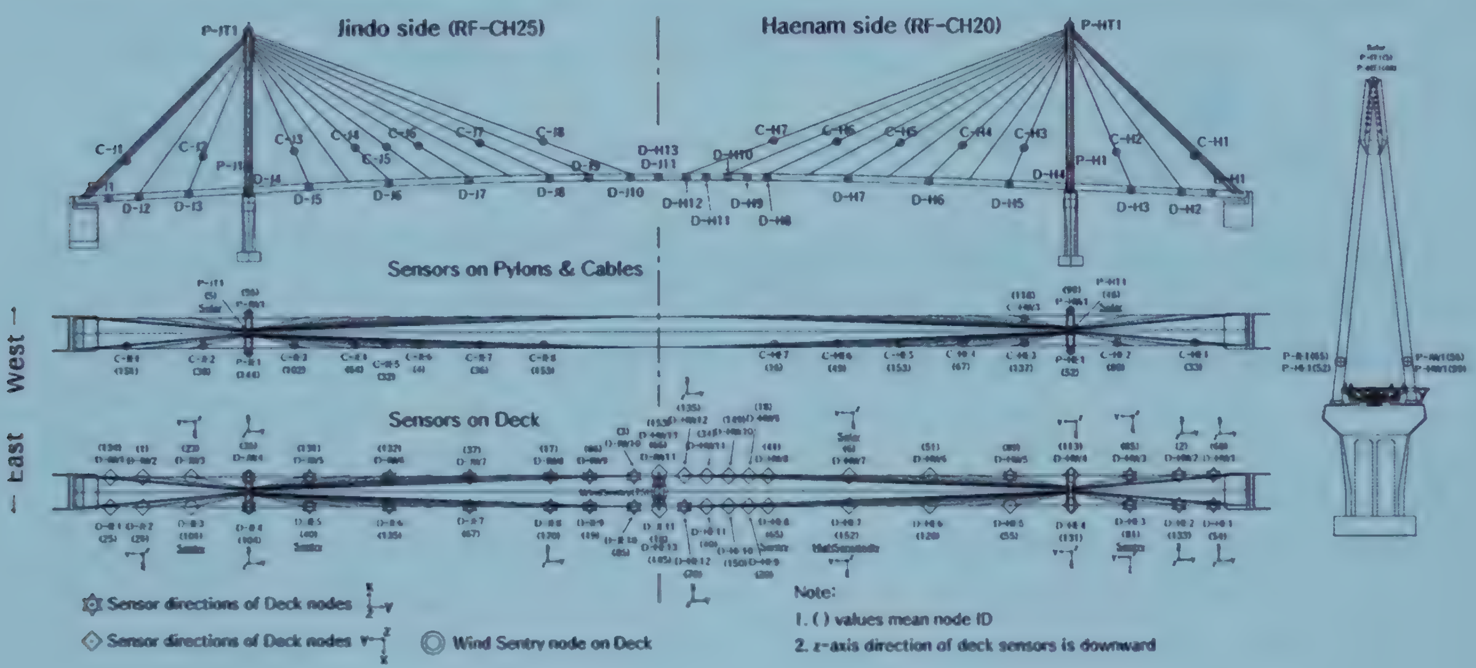
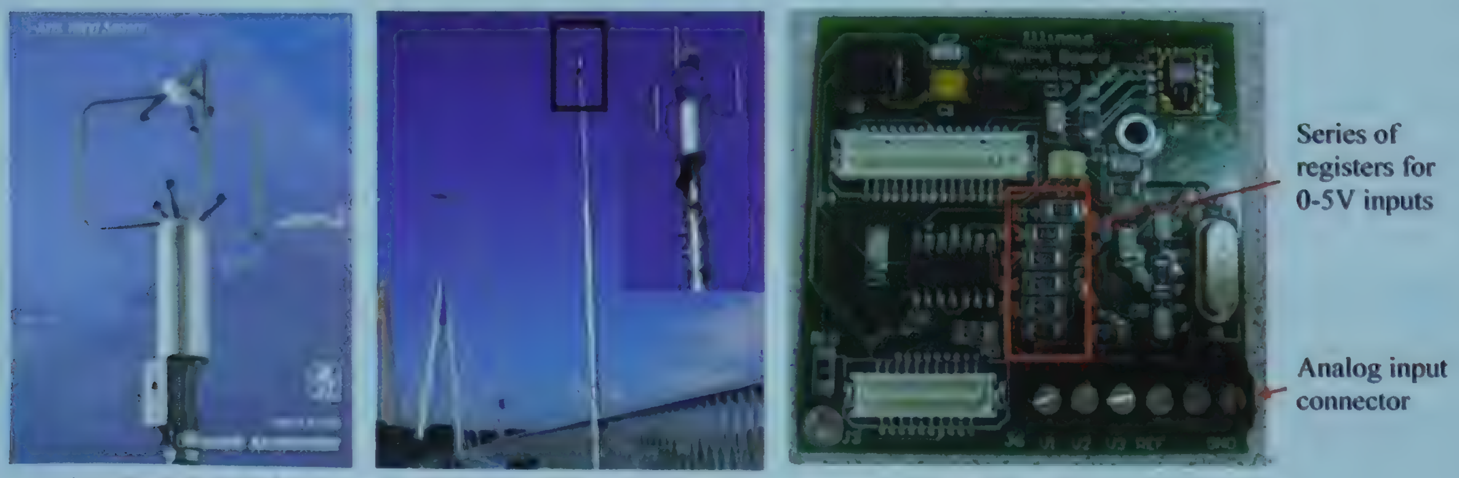

Jindo side (RF-CH25)
Haenam side (RF-CH20)
Sensors on Pylons & Cables
Sensors on Deck
West →
← East
Sensor directions of Deck nodes
Sensor directions of Deck nodes
Wind Sentry node on Deck
Note:
1. ( ) values mean node ID
2. z-axis direction of deck sensors is downward

Fig. 13.3 Sensor location

A 3-D ultra-sonic anemometer shown in **Fig. 13.4** has been incorporated into the Wireless Sensor Network (WSN) for reliable wind monitoring of the Jindo Bridge. Because of its high resolution (wind speed: 0.01 m/s, wind direction: 0.1 degree) and good accuracy (wind speed: ±1%, wind direction: ±2 degrees).


Series of registers for 0-5V inputs
Analog input connector

(a) Model 81000 (b) On Jindo Bridge (c) SHM-W board

Fig. 13.4 3D ultra-sonic anemometer and SHM-W board

Data Acquisition, Transmission, Processing and Management

The key features of these services include Remote Sensing, a remote data measurement application; Decentralized Data Aggretation, data measurement and on-board computation to calculate the correlation functions for decentralized local groups; imote2comm and autocomm, an automatic terminal tool to interact between the base station PC and a gateway node; Vbat, a command for checking a leaf node's battery level; TestRadio, a tool for assessing radio communication quality, and many others.

The continuous and autonomous monitoring service critical for this deployment is AutoMonitor, an autonomous SHM network management application, which combines RemoteSensing, ThresholdSentry, and SnoozeAlarm (Rice and Spencer 2009). SnoozeAlarm is a strategy that allows the network to sleep most of the time, thus improving energy efficiency and allowing long-term system deployment. To wake the network for an important event, the ThresholdSentry application defines a specified number of the leaf nodes as sentry nodes. The sentry nodes wake up at predefined times and measure a short period of acceleration or wind data. When the measured data exceeds a pre-defined threshold, the sentry node sends an alarm to the gateway node, which subsequently wakes the entire network for synchronized data measurement. In this way, AutoMonitor enables the automatic, continuous monitoring with reduced power consumption.

The AutoMonitor application employs multiple threshold levels for the Jindo Bridge SHM system. When a single threshold level is used to define the maximum number of events during given period, if the threshold level is too high, the data measurements would rarely if ever occur. If too small a threshold level is used, the number of events can be exhausted with small structural responses, hence possible strong vibrations during remaining period cannot be captured. The multiple threshold levels, having separate maximum number of events, make it possible to measure both rare but strong responses and frequent but low-level ambient vibration during a given period, which enables more effective SHM.

Another feature of the AutoMonitor application is a wind threshold sentry. The wind sentry node is equipped with SHM-W sensor board with ultrasonic anemometer. Strong winds, such as those that occur during typhoons, induce large structural responses. Therefore, the wind sentry will trigger network sensing when the velocity exceeds a threshold. In this deployment, both vibration- and wind-based sentry nodes have been installed for the AutoMonitor application. In summary, the final software version is the AutoMonitor application having both data measurement and on-board computational functionality, a multiple threshold triggering strategy using vibration/wind sentries pursuing energy efficiency using SnoozeAlarm.

Table 13.4 shows the list of the network parameters and values for the initial state and the updated state.

Table 13.4 shows the list of the network parameters

Category	Parameter	Initial state	Updated state
RemoteSensing	Number of *RemoteSensing* events	1 per day	4 per day
	Time synchronization wait time	30 sec	30 sec
	SENSING START DELAY	15,000	50,000
	NODE SENSING START DELAY	1,000	1,500
	Sampling frequency	50 Hz	10 Hz
	Channels sampled	3	3
	Number of data points	1,000	5,000
Snooze Alarm	*SnoozeAlarm* wake/listen time	750 ms	750 ms
	SnoozeAlarm sleep time	15 sec	15 sec
	SnoozeAlarm duty cycle	4.76%	4.76%
ThresholdSentry	*ThresholdSentry* check interval	20 min	10 min
	ThresholdSentry sensing time	10 sec	10 sec
	Threshold value for vibration sentry	10 mg	10 mg, 50 mg
	Threshold value for wind sentry	N/A	3 m/sec, 8 m/sec
Watchdog Timer	Period	10 min	60 min
	Update	1 min	60 min

13.5.3 *SHM description of Rion-Antirion Bridge, Greece [14]*

Bridge Details

The Rion-Antirion Bridge is a 4-tower. 5 span cable-stayed structure. The continuous composite deck has total length of 2252 m, with 3 main spans of 560 m and side spans of 286 m, and is suspended by 4 concrete towers, with total height of 189 up to 227 m, through 368 cables with total length from 79 up to 295 m. At each far end of the deck, a steel rotating frame (RF) supports the structure allowing longitudinal movement that is accommodated by special designed expansion joint. Furthermore, at tower and RF locations, the deck is transversally restrained through a fusing steel element that releases the deck when the transverse load, on each element, exceeds ±10.500/±3.400 kN (tower/RF).

Parameters to be Monitored and Location Selection

The current architecture of the monitoring system, **Fig.13.5** can be divided into 4 different levels namely, (a) Sensors (b) Power supply and signal transferring (d) Digitalization, acquisition and signal processing and (d) Communication network and data management which are explained in detail in the upcoming sections. Largely, the monitoring system consists of:

i) Weather stations for monitoring of wind direction, velocity, ambient temperature, humidity and air pressure,
ii) load cell on fuse restrainer,
iii) accelerometers for monitoring wind effects on deck,
iv) accelerometers for characterization of earthquake motion,
v) accelerometers for monitoring earthquake effects on deck,

vi) accelerometers for monitoring earthquake effects on towers,

vii) monitoring of expansion joint openings,

viii)detection of water inside tower legs,

ix) road temperature,

x) characterization of wind effects on stay cables:

- accelerometers for monitoring vibrations induced by traffic and wind,
- mono strand load cells for monitoring of tension in the stays,
- displacement transducers for monitoring of the vibration amplitudes,
- strain gauges for monitoring of internal stresses in gussets (connection between stay anchorage and deck)

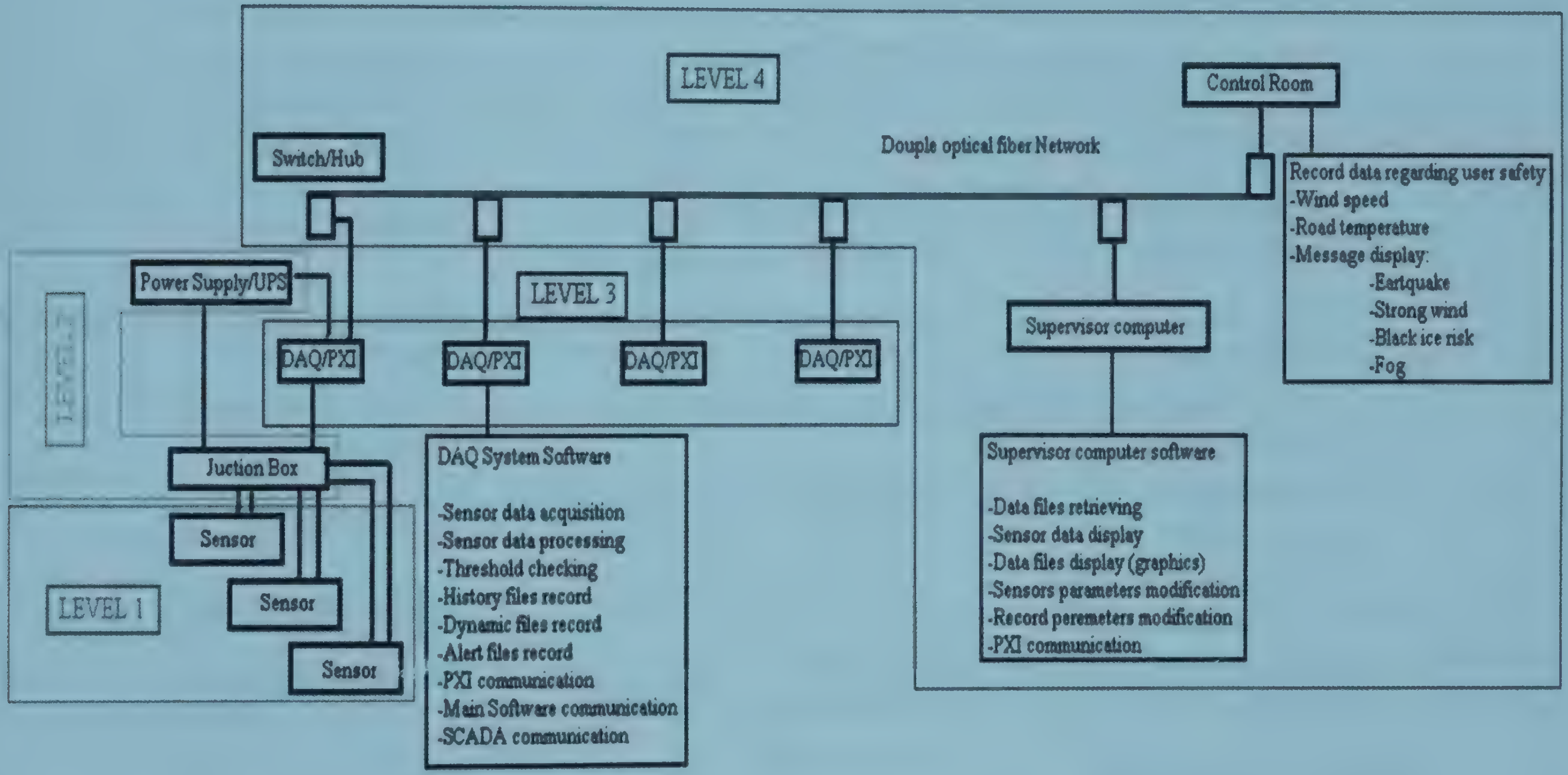

LEVEL 4
Control Room
Switch/Hub
Douple optical fiber Network
Record data regarding user safety
-Wind speed
-Road temperature
-Message display:
-Eartquake
-Strong wind
-Black ice risk
-Fog
Power Supply/UPS
LEVEL 3
LEVEL 2
DAQ/PXI
DAQ/PXI
DAQ/PXI
DAQ/PXI
Supervisor computer
Juction Box
Sensor
Sensor
Sensor
LEVEL 1
DAQ System Software
-Sensor data acquisition
-Sensor data processing
-Threshold checking
-History files record
-Dynamic files record
-Alert files record
-PXI communication
-Main Software communication
-SCADA communication
Supervisor computer software
-Data files retrieving
-Sensor data display
-Data files display (graphics)
-Sensors parameters modification
-Record peremeters modification
-PXI communication

Fig. 13.5 Overview of structural monitoring system

Sensors:

A large variety of sensors as shown in **Fig. 13.6** and listed in **Table 13.5** were incorporated to the monitoring system in order to record the response of various structural elements under different loading conditions.

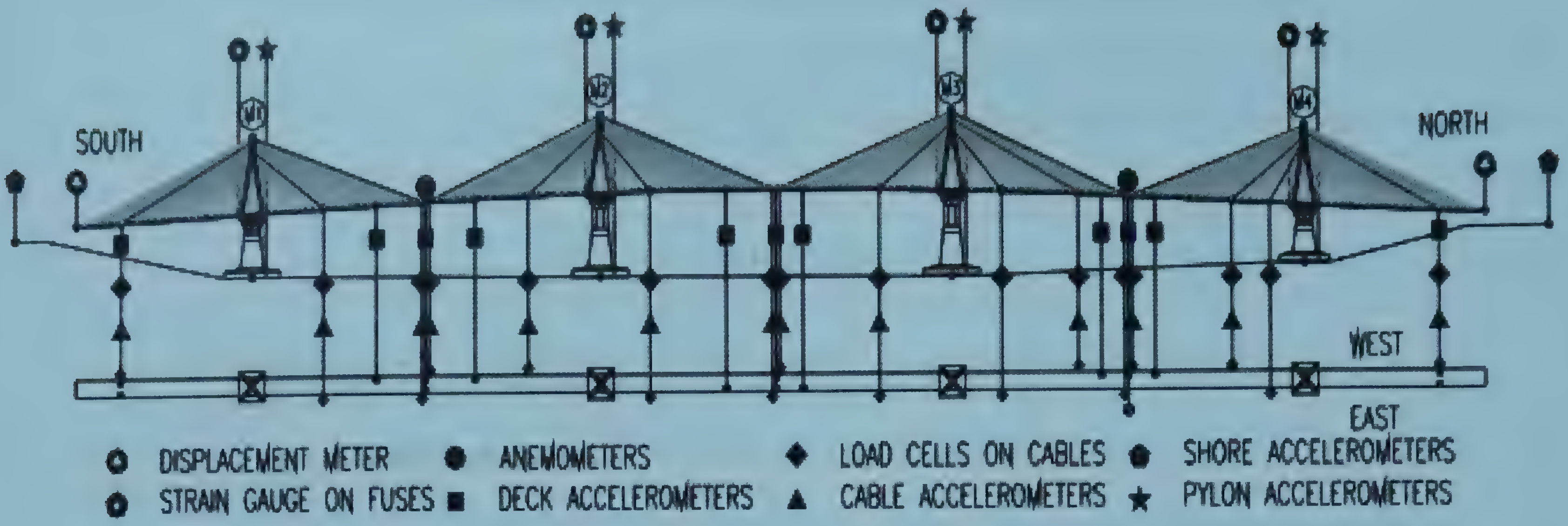

SOUTH
M1
M2
M3
M4
NORTH
WEST
EAST
DISPLACEMENT METER
ANEMOMETERS
LOAD CELLS ON CABLES
SHORE ACCELEROMETERS
STRAIN GAUGE ON FUSES
DECK ACCELEROMETERS
CABLE ACCELEROMETERS
PYLON ACCELEROMETERS

Fig. 13.6 Layout of Structural Monitoring System

Table 13.5 Sensor Description

Sensor	Quantity	Expected range of values	Actual sensor range	Monitored phenomenon
3D anemometers	2	0-50 m/sec	0-60 m/sec	Wind intensity
Temperature and Humidity sensor	2	50° C/0-100%RH	-50°C, up to 50°C/ 0-100% RH	Thermal loading
3D Pylon accelerometers	12	±1.9g (top) ±1.0g(base)	±20g(top) ±3g(base)	Pylon vibration (Earthquake/wind)
1D/3D Deck accelerometers	3/12	±2.7g	±3g	Deck vibration (Earthquake/wind)
3D Ground accelerometers	2	±0.48g	±3g	Earthquake
3D Cable accelerometers	13	-	±3g	Cable vibration Wind
Monostrand load of cables	16	0 up to 75% F_{GUTS} (199 kN)	0-320 kN	Cable load variation (Wind/Earthquake/Balance)
Magnetic distance meter	2	+1260/-1150 mm	3 m	Expansion joint opening (Earthquake/Balance/Thermal)
Strain gauges (full bridge)	4	±10500 kN	±1500$\mu\varepsilon$ ±17000 kN	Wind induced lateral load
Road temperature sensors	4	-	-50°C, up to 50°C	User safety (black ice risk)
Deck temperature sensors	5	-	-10°C, up to 80°C	Thermal loading

Data Acquisition, Transmission, Processing and Management

Due to the size of the monitored structure (more than 2500 m between extreme sensors), 4 different digitalization and acquisition units (DAQs) are installed, each on one tower. The

DAQs are located in controlled temperature shelters and are specially designed to be operable in harsh environments. The following operations are performed in each DAQ:

- Low pass hardware filtering at 10 kHz
- Digitalization at 500 Hz
- Conversion of signal to engineering units
- Continuous threshold checking
- Data file creation and real time transmission of selected values

A significant aspect, necessary for any further analysis of the recorded signals, is synchronization. In current architecture all the DAQs are synchronized with a server through SNTP protocol.

The communication of each DAQ with the supervisor computer (SE) and with the Control Room of the Bridge is performed through an optic fiber network that is installed along the deck. Thus, the collected data are retrieved, evaluated and permanently stored to dedicated media inside the operation building. Additionally, through the SE, the end user can overview all the measurements (real time) and access/modify all record parameters (thresholds, scaling, acquisition parameters etc.) that can be adjusted to each monitored event.

Evaluation of Structural Performance

Two main categories of data files are created by the monitoring system:

a) History files (0.5 sec averaged values recorded every 30 sec, except wind speed and direction that are 10' average)

b) Dynamic files (High sampling frequency at 100 Hz with 60 sec duration): The History files are created continuously while the dynamic files are recorded every 2 hours (Automatic) or when particular threshold has been over passed (Alert files) or even at user demand (Request).

All types of files (History/Automatic/Alert/Request) are very useful in order to understand the actual bridge response. Each of them can be used for different analysis purposes. In particular, the History files are very useful in:

- Characterization of Environmental conditions, (wind, temperature)
- Estimation of deck creep and shrinkage
- Evaluation of static impact of wind loads
- Identification of potential loss of cable load from overall force distribution

13.5.4 *SHM Description of Chaibu Bridge, China [15]*

Bridge Details

This is a two-tower cable-stayed bridge carrying six lanes of traffic over the Fuchun River. It is located on the highway between the Meirong and the Fengchuan in the Tonglu County, China. The construction of the bridge began in April 2014 and is expected to complete in December 2017. The cable stayed bridge has a total length of 412 m (106 m + 200 m + 106 m). There are 104 cables divided into four central cable plane parts and arranged in twin towers in total. The decks are composed of cast-in-situ concrete box girder by cantilever method. Each cable tower is a single column tower, and the main tower is 64 m high using box-section. The pier is double thin-wall structure with rectangle section. The connection of bridge's girder, tower, and pier is rigid joint.

Parameters to be Monitored and Location Selection

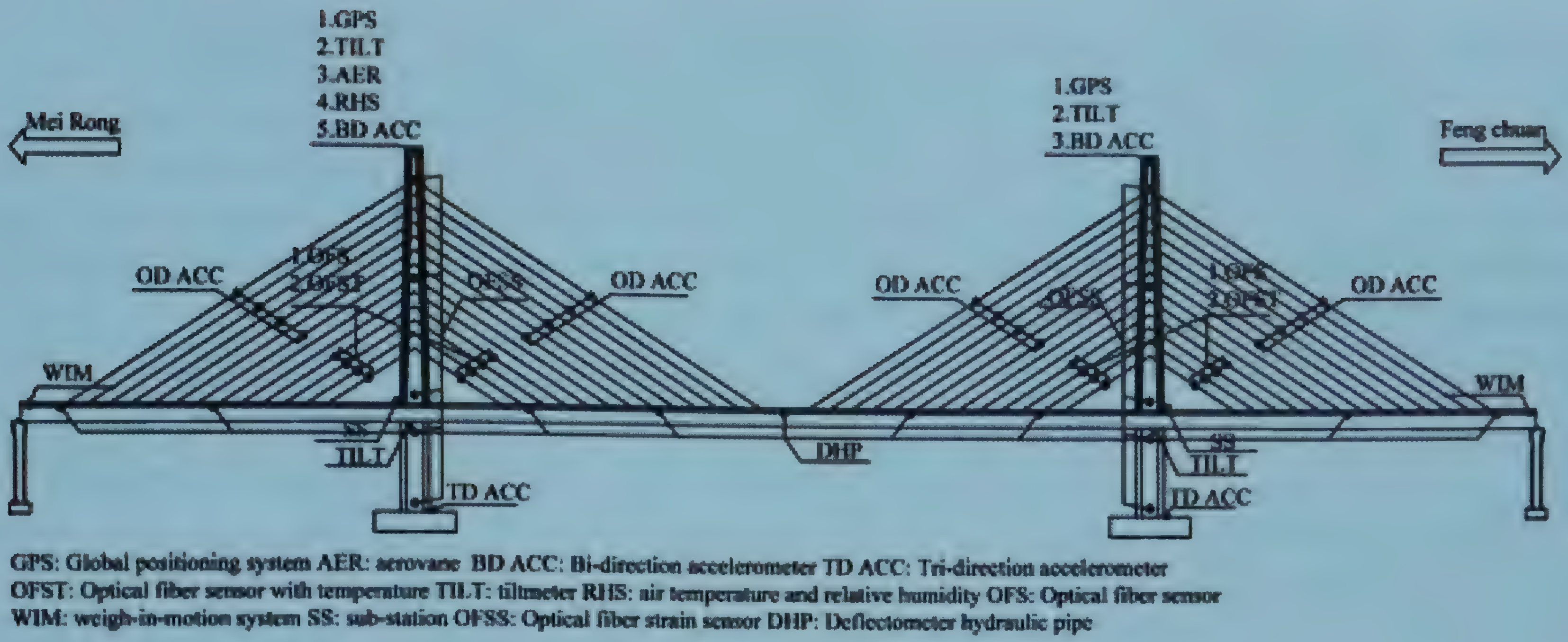
The whole SHMS consists of four subsystems, including sensor subsystem, data acquisition and transmission subsystem, database management subsystem, detection and evaluation subsystem.

Sensor Subsystem

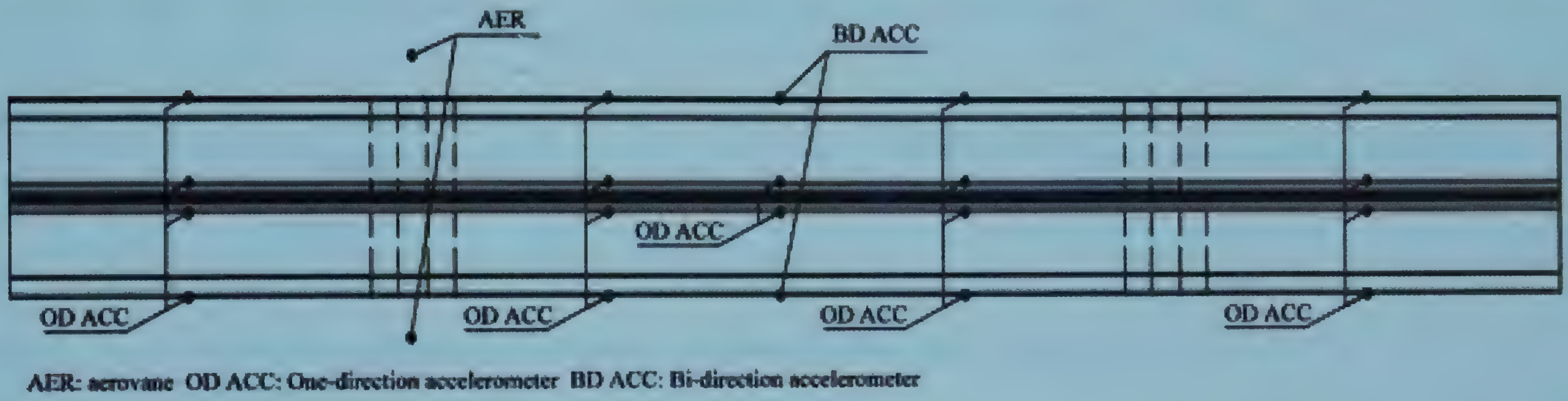
The optical fiber strain sensors are used to monitor the changes of strain. Additionally, weigh-in-motion system (WIM), one-direction accelerometers (ACC), and global positioning system (GPS) are installed on the bridge to measure the load of different vehicles, acceleration and displacement signals of structure in time domain. In this monitoring system, AER is established to record the atmosphere temperature and humidity for gaining the change law of some physical parameters and revising the test data. The **Fig. 13.7** shows the details of sensors layout and the **Table 13.5** lists the number of sensors

Data Acquisition and Transmission Subsystem

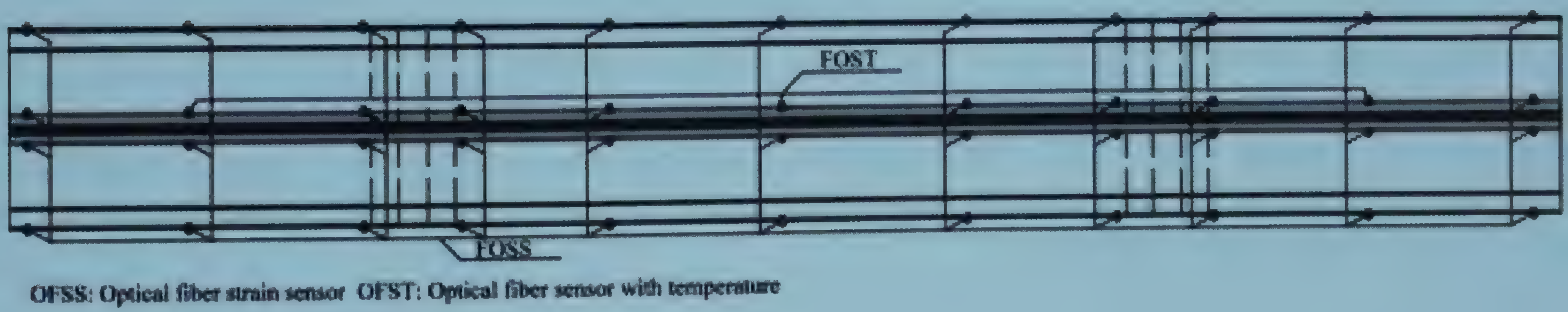
The monitoring signals were divided into static and dynamic signals by the frequency of signals. Based on distributed static signal with RS485 serial bus, the input/output (I/O) data acquisition system is designed. As for the high sampling rate dynamic sensors, it takes the IEEE1588 standard to ensure the acquisition system clock synchronization. The distributed data acquisition system uses industrial Ethernet for Dynamic signals transmission. Besides, LabVIEW, a product of NI Inc., USA, is selected to be the software to write the program for the collection of sensor signals. At present, the signal transmission technologies can be classified into two categories that are the wired transmission and the wireless transmission. In this SHMS, these two signal transmission technologies are combined for transferring. Firstly, the signals like OFSS and OFST are transmitted to the industrial computer installed on the bridge by wired transmission technologies. And then the data transmit to the server installed in the station building by wireless transmission technology.


1.GPS
2.TILT
3.AER
4.RHS
5.BD ACC
Mei Rong
Feng chuan
1.GPS
2.TILT
3.BD ACC
OD ACC
1.OFS
2.OFST
OFSS
OD ACC
OD ACC
OFSS
1.OFS
2.OFST
OD ACC
WIM
WIM
SS
TILT
TD ACC
DHP
SS
TILT
TD ACC
GPS: Global positioning system AER: aerovane BD ACC: Bi-direction accelerometer TD ACC: Tri-direction accelerometer
OFST: Optical fiber sensor with temperature TILT: tiltmeter RHS: air temperature and relative humidity OFS: Optical fiber sensor
WIM: weigh-in-motion system SS: sub-station OFSS: Optical fiber strain sensor DHP: Deflectometer hydraulic pipe

(*a*) Layout of sensors on elevation view


AER
BD ACC
OD ACC
OD ACC
OD ACC
OD ACC
OD ACC
AER: aerovane OD ACC: One-direction accelerometer BD ACC: Bi-direction accelerometer

(*b*) Layout of accelerometers


FOST
FOSS
OFSS: Optical fiber strain sensor OFST: Optical fiber sensor with temperature

(*c*) Layout of optical fiber sensors

Fig. 13.7 Details of Sensors Layout

Database Processing and Management Subsystem

A database based on the SQL Server 2008 is developed as a module of this SHMS. The entire database is composed of the static and dynamic database. The static database is used for storing static information, e.g. geography information of the bridge site, detailed design information of the bridge, layout of sensors. The dynamic database is related to the dynamic sampling rate. To prevent the shortage of the memory, some measures should be taken for database management when the memory comes to a threshold set by bridge manager. Meantime, computer-user interface designs for interactive requirements based on B/S model. Because the B/S model interface lowers the requirement of client configuration and improves systems operability. Some safety means are used in the SHMS to prevent intrusion of the database, such as password, virus resistance. The connection between different subsystems is shown in the **Fig. 13.8**

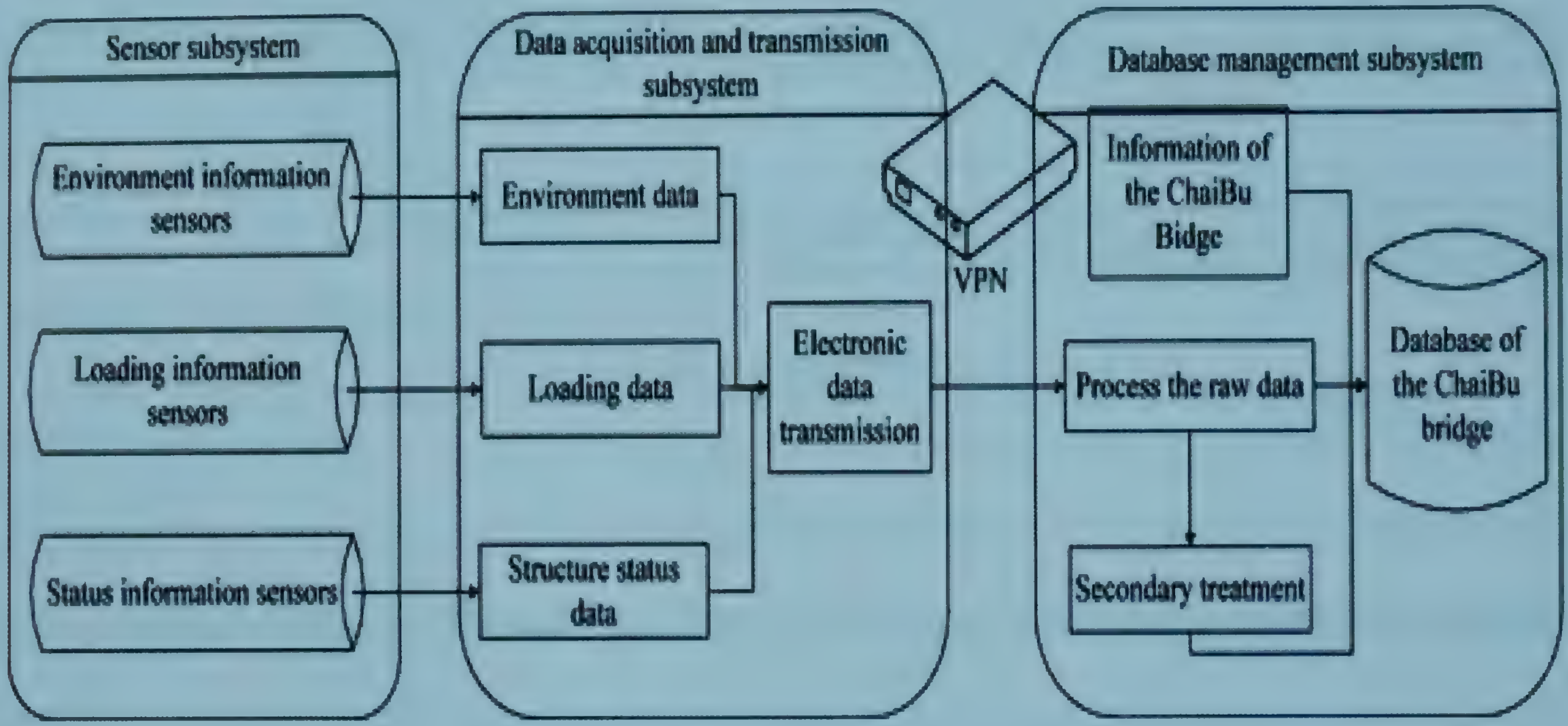

Sensor subsystem
Environment information sensors
Loading information sensors
Status information sensors
Data acquisition and transmission subsystem
Environment data
Loading data
Structure status data
Electronic data transmission
VPN
Database management subsystem
Information of the ChaiBu Bidge
Process the raw data
Secondary treatment
Database of the ChaiBu bridge

Fig. 13.8: The connections between different subsystems

Evaluation Subsystem of Structural Performance

The detection and evaluation subsystem in the SHMS is to detect damage and evaluate the condition of the bridge. The typical damage detection methods, such as frequency response model, modal curvature method, and data fusion technique for damage detection, are integrated into this subsystem and can be automatically operated by utilizing the measured data. As for the FEM, the sensitivity analysis method is employed to automatically update the FEM. For example, material elastic modulus and pre-stressed force will change with time. With several iterative cycles, the FEM can be used for analyzing the structure condition

CHAPTER - 14

AESTHETIC CONSIDERATIONS

14.1 Introduction

The synthesis of structural system and visual form is near perfect in the case of Cable Stayed Bridges, which derive their aesthetics from their pleasing appearance, economy of material usage and lack of any embellishments for purposes other than purely structural. The flow of forces in the structure are self-evident and that is part of the enchanting appearance of this concept of bridge construction. All the members are predominantly subjected to axial forces, with the pylon and the deck under compression and the cables under tension which leads to a very efficient structural system. The low consumption of materials in the light superstructure results also in saving of materials and costs of the foundations thereby enhancing the sustainability aspects. Aesthetics of Cable stayed bridges require particular attention to its integration in urban spaces.

By their very nature the design and construction of Cable stayed bridges requires skills of a high caliber apart from an intimate knowledge of structural behavior. They are a popular option for long bridges when the main spans are upwards of 200 m, **Fig. 4.1 (Chapter 4)**. Due to its high degree of indeterminacy the use of high speed computing and appropriate softwares become a basic requirement for its analysis

14.2 Structural System and Structural Elements

Aesthetics of Cable Stayed Bridges is closely related to:

- Tower shape,
- Arrangement of stays
- Structural form of the deck
- Proportion of elements below the deck
- Integration with the surrounding environment
- Lighting

Aesthetics of cable stayed bridges must be seen differently as compared to conventional bridges. The additional elements of pylons and stay cables introduced in the structure have a huge visual impact. The size of the pylon with its enormous height becomes the most dominating feature of the bridge, and, most of the time even the landscape. The closely spaced stay cables provide a transparent veil that gives the bridge an element of delicacy. The appeal of the cable-stayed bridges is not limited to engineers but has evinced great interest of the public at large. The interest generated in the media for the Signature Bridge in Delhi, **Fig. 4.8 (Chapter 4)** is a case in point.

14.3 The Tower

The tower shape is dependent on a number of factors such as one/two planes of stays, the foundation size to which must be transferred all loads and the structural elements that keep the second order effects (buckling) under control. The straight towers of **Fig. 14.1**, Vidyasagar Setu, Kolkata and the 'cranked' towers of **Fig. 14.2** Atal Bridge @Basholi have acquired their shapes to suit the foundation size.

The horizontal struts connecting the two tower shafts provide relief from second order effects. The tower is under heavy compression as it has to balance the vertical component of the tensions of the stays. The inclination of the shafts constituting the towers is an important aspect of design. The two shafts can be kept separated, **Fig. 14.1**, or, fused together at some height and extended as a single vertical element forming an inverted Y shape, **Fig. 14.2**.

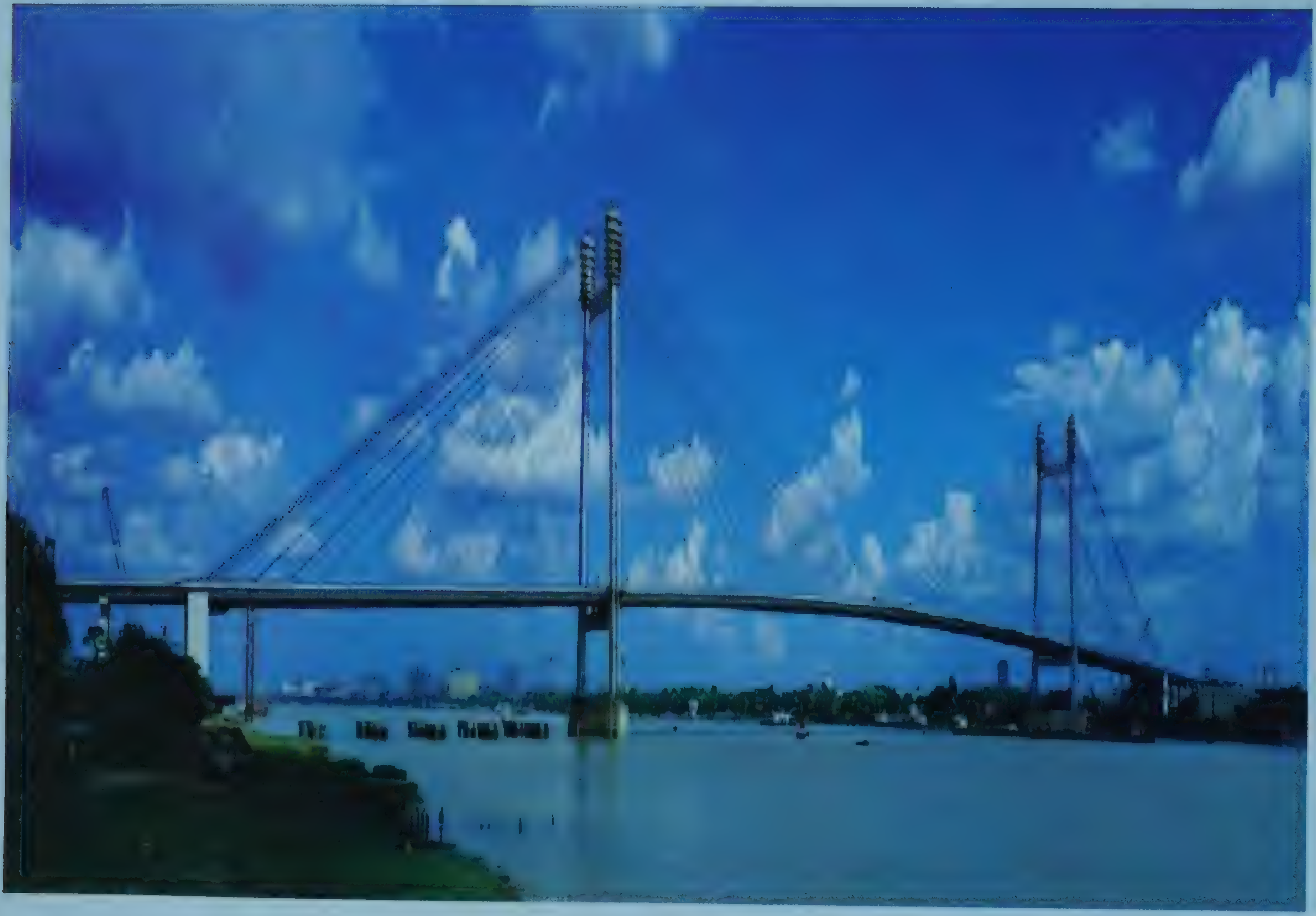


Fig. 14.1. Vidya Sagar Setu: Tower shape is governed by foundation size.

Note the connecting beam at the top to reduce second order effects

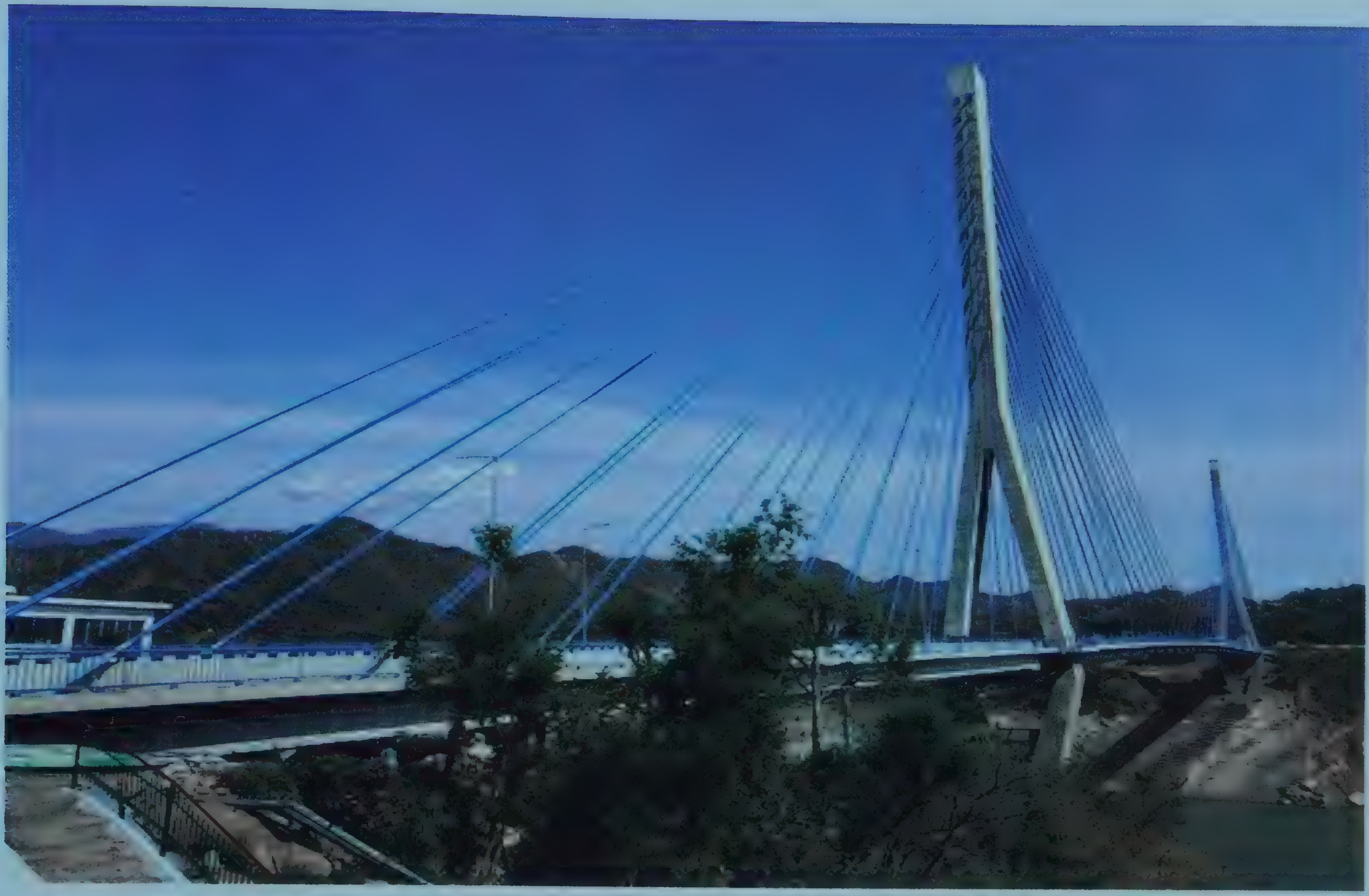


Fig. 14.2. Atal Bridge, Basholi. Tower shape is governed by foundation size

Note the merging of towers towards the top to reduce second order effects

14.4 Arrangement of stays

It is well recognized that the harp system of stay cable arrangement, **Chapter 3**, is the most preferable one from the point of view of aesthetics. Even for two planes of stays in a harp arrangement they do not intersect whatever be the position and angle of the viewer.

14.5 The Deck

The deck is usually very thin with a depth to main span ratio sometimes as low as 250 and this characteristic of sleekness contributes greatly to the overall aesthetics. The constant depth throughout the bridge length with closely spaced supports provided by the stay cables adds further value to the appearance.

14.6 Proportion of elements below the deck

From a purely aesthetic point of view, the proportion of the structure should be such that the pier height should be much smaller than the spans of the superstructure that it supports. Due to the long spans involved in the cable stayed bridge constructions this requirement is fulfilled automatically. Also, in Indian Conditions there is rarely a requirement of big heights that are necessary for the introduction of clearances for ocean liners and big ships.

14.7 Integration with the surrounding environment

The Cable stayed bridge can have numerous variations of the basic bridge type which makes it easy for it to fit into most surroundings once there is a requirement of a long span. The integration of the bridge into its environment or landscape, or the urban or semi-urban situation remains the most important consideration in relation to aesthetic design, but it seldom necessitates any structural modifications An example that shows how the bridge has blended beautifully in the hilly terrain is shown in **Fig. 14.3** Atal Bridge @Basholi.

14.8 Lighting

Lighting of the cable-stayed bridge at night must form an integral part of the design. This is in addition to the lighting of the deck carriageway which is a purely functional requirement. The closely spaced stay cables are of great interest from the point of view of aesthetics as they provide a translucent background on which lighting can be projected with great effect. The light source for aesthetic lighting can be on the central verge of the deck, on the footpaths or at the top of the tower. These options and a combination of them provide ample opportunities for creativity as far as projection of lights on the stay cables is concerned. The lighting provided for aesthetic purposes should ensure that it does not create a glare for the drivers and also does not interfere with aircraft navigation.

Lighting of the cable stays with LED lights can greatly enhance the appearance of the bridge at night. Excellent example of this can be seen in **Fig. 14.4** (Chambal Bridge in Kota) and **Fig. 14.5** (Mandovi Bridge, Goa). They have made a new tradition called "structural art". Superimposing laser images on the stays is another area of expertise. The images can be changed from time to time and New Zuari Bridge, **Fig. 14.6.**

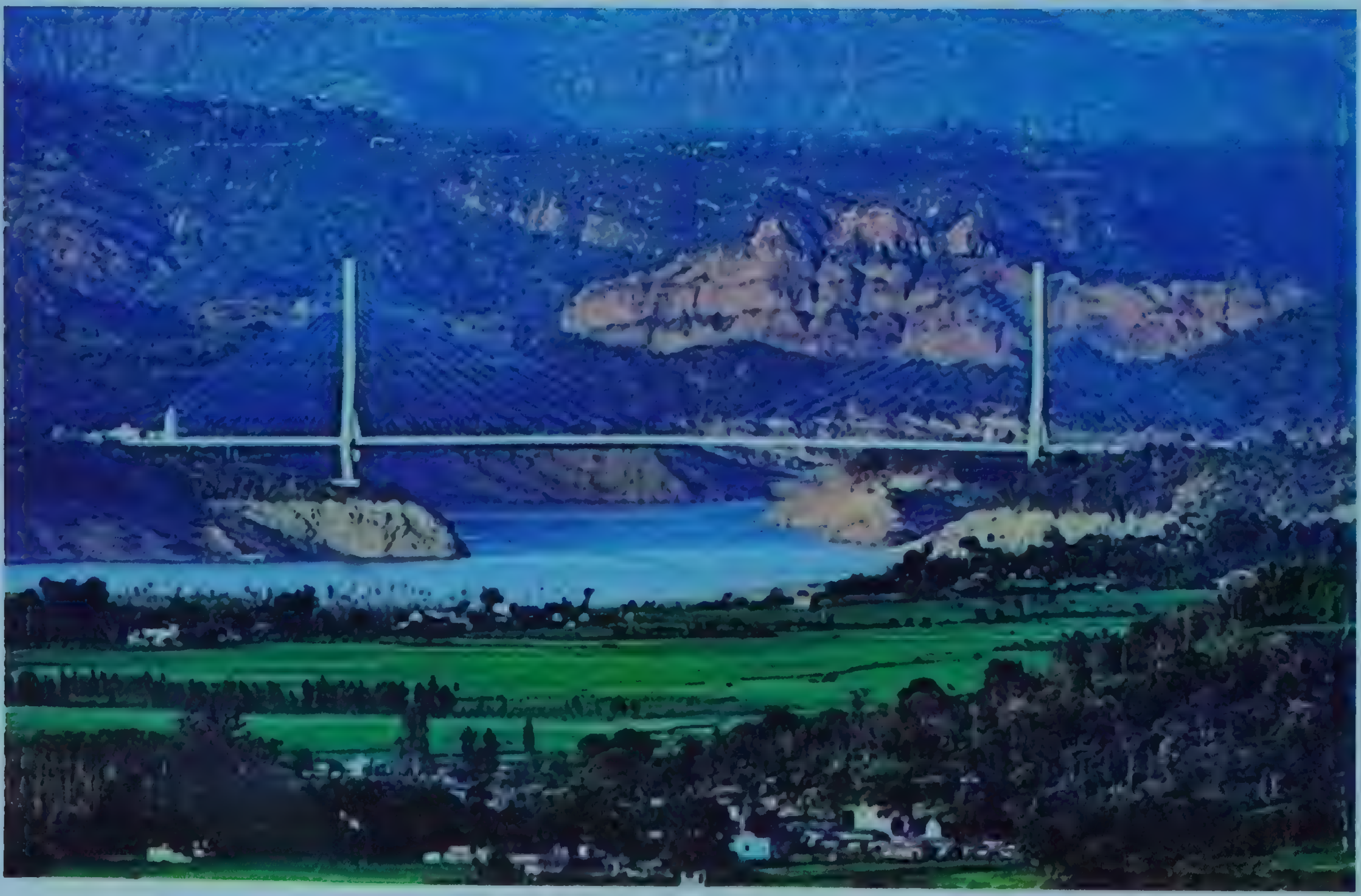


Fig. 14.3 Atal Bridge, Basholi. Aesthetic blending into the hilly terrain

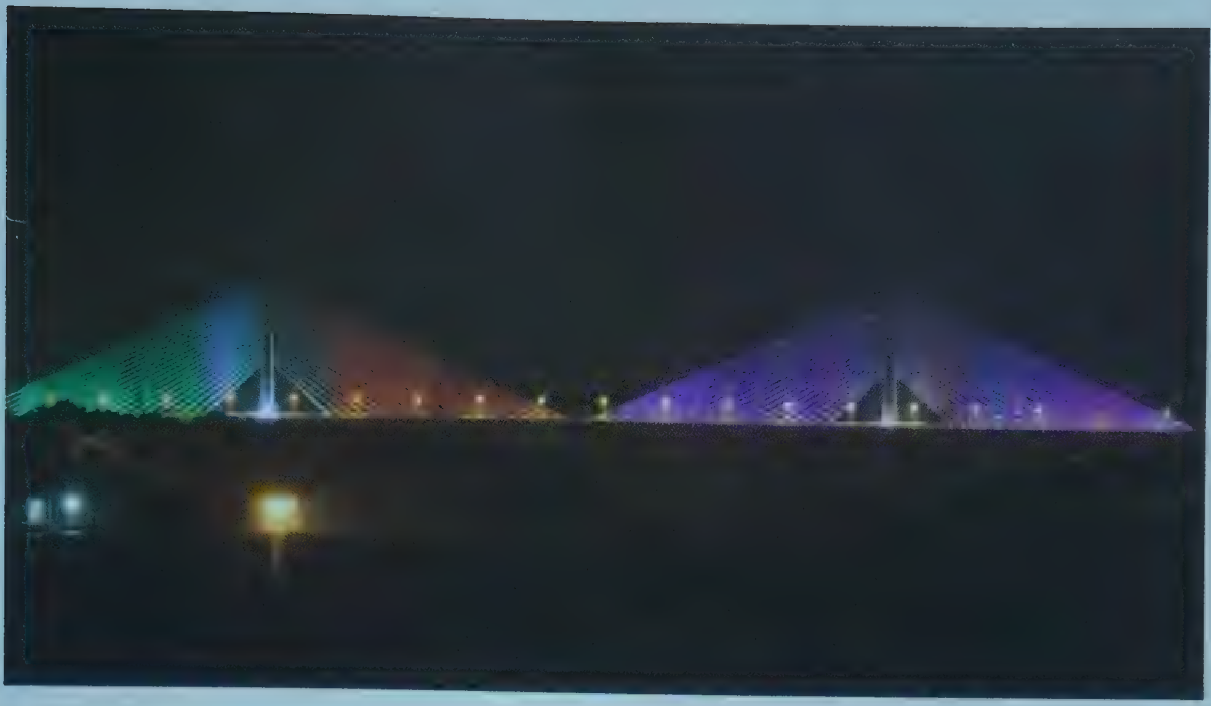


Fig. 14.4 Chambal Bridge in Kota

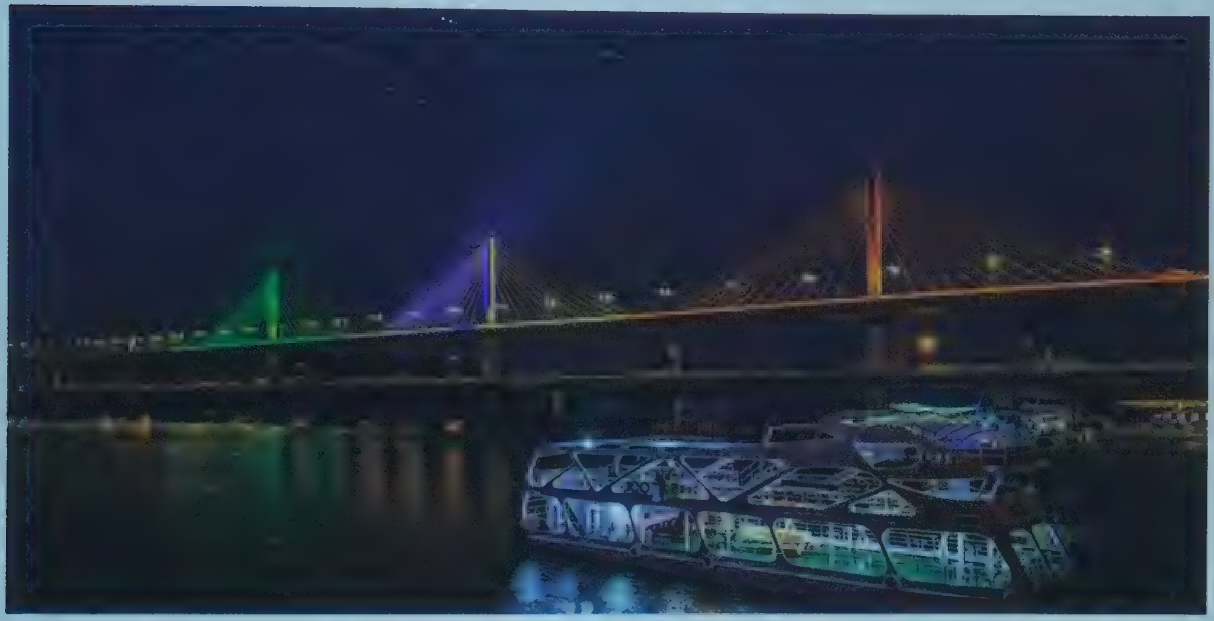


Fig. 14.5 Mandovi Bridge, Goa

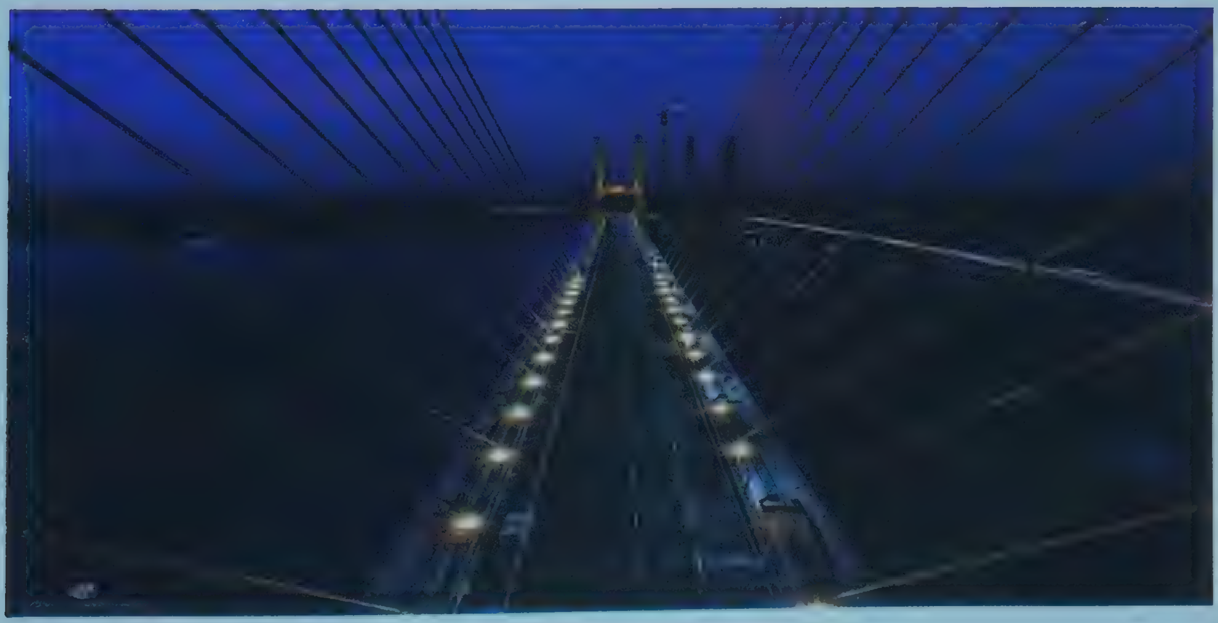


Fig. 14.6 New Zuari Bridge, Goa

Annexure A2
(Pertaining to Chapter 7)

APPROACH TO DESIGN OF STAY CABLES AS PER INTERNATIONAL GUIDELINES

This Annexure presents philosophy and Methods for the design of cable for cable stayed bridges and extradosed bridges adopted by Japanese Code *"Specifications for Design and Construction of Cable Stayed Bridges and Extradosed Bridges"*, Setra – Stay Cable Recommendation and PTI Recommendations which are the codes available for design cable supported bridges. Also, recommendation of the FIB 89 are presented at the end.

I. Specifications for design and construction of cable stayed bridge and extradosed bridge - JPCI

As per Kasuga in his paper **[1]**, when designing structures that are reinforced using stays, rather than defining in advance whether the bridge will be a cable-stayed bridge or an extradosed bridge and then determining the allowable stress for the stays, the more rational approach would be to design the stays by focusing on the stress change caused by live loads that affect fatigue. This would make it possible to design each stay separately and enable the allowable stress to be set individually for each stay. From the outset, unlike suspension bridges, the stress change on a cable stayed bridge will differ depending on the stays, so it is not rational to define the allowable stress using a single value of 0.4 f_{pu}. This knowledge is reflected in JPCI specifications.

The specification allows two kinds of design method. One is normal fatigue design using fatigue load and design lifetime of a bridge (Method A). However, it is usually difficult to estimate the amount of future traffic and heavy trucks, especially in local roads. In that case, another approximation method using stay-cable stress change owing to designed vehicular live loads is introduced (Method B). **Fig. A2.1** shows the comparison of Methods A and B. As per Method B the stress variation in the cable due to design live load $\Delta\sigma_L$ is determined, based on $\Delta\sigma_L$, permissible stresses in cables at service loads is determined to carry our serviceability check. The above relation between the stress changes due to live and permissible stress is given for both prefabricated wire type and strand type cables.

Method B is defined such that it ensures adequate safety in comparison with bridges designed using Method A.

Fig. A2.1 shows fatigue stress range ($\Delta\sigma_{2E6}$) in the cable (Based fatigue design performed for 3 bridges). This fatigue stress range is compared to fatigue strength (f_{scrd}) divided by safety factor (γ_b).

For a strand stay cable fabricated on site using wedges, based on a system with fatigue strength of at least 120 N/mm^2 at 0.6 f_{pu} or at least 200 N/mm^2 at 0.4f_{pu}. In this situation, γ_b is 1.4. The shaded section of the figure is the range determined by Method A ($\Delta\sigma_{2E6}$) with the fatigue design conditions indicated above. This is two-thirds of the $\Delta\sigma_L$, as prescribed by Method B, there is a safety factor of around 2.0 with respect to f_{scrd} /γ_b.

For a galvanised wire stay cable made at a factory as a cold-cast cable or as a cable with buttonhead anchorages, the relationship between the f_{scrd}/γ_b and the $\Delta\sigma_{2E6}$ estimation line is shown in **Fig. A2.1**, based on a system with fatigue strength of at least 180 N/mm^2 at 0.6 f_{pu} or at least 230 N/mm^2 at 0.4 f_{pu}, and like the figure for the cable fabricated on site. It can be seen from the figure that the factory-made cable also has a safety factor of around 2.0 with respect to f_{scrd}/γ_b.

Therefore, in the stays designed by Method B, Δ_{sL} is determined to require a safety factor of about 2.0 for $\Delta\sigma_{2E6}$ with respect to f_{scrd}/γ_b.

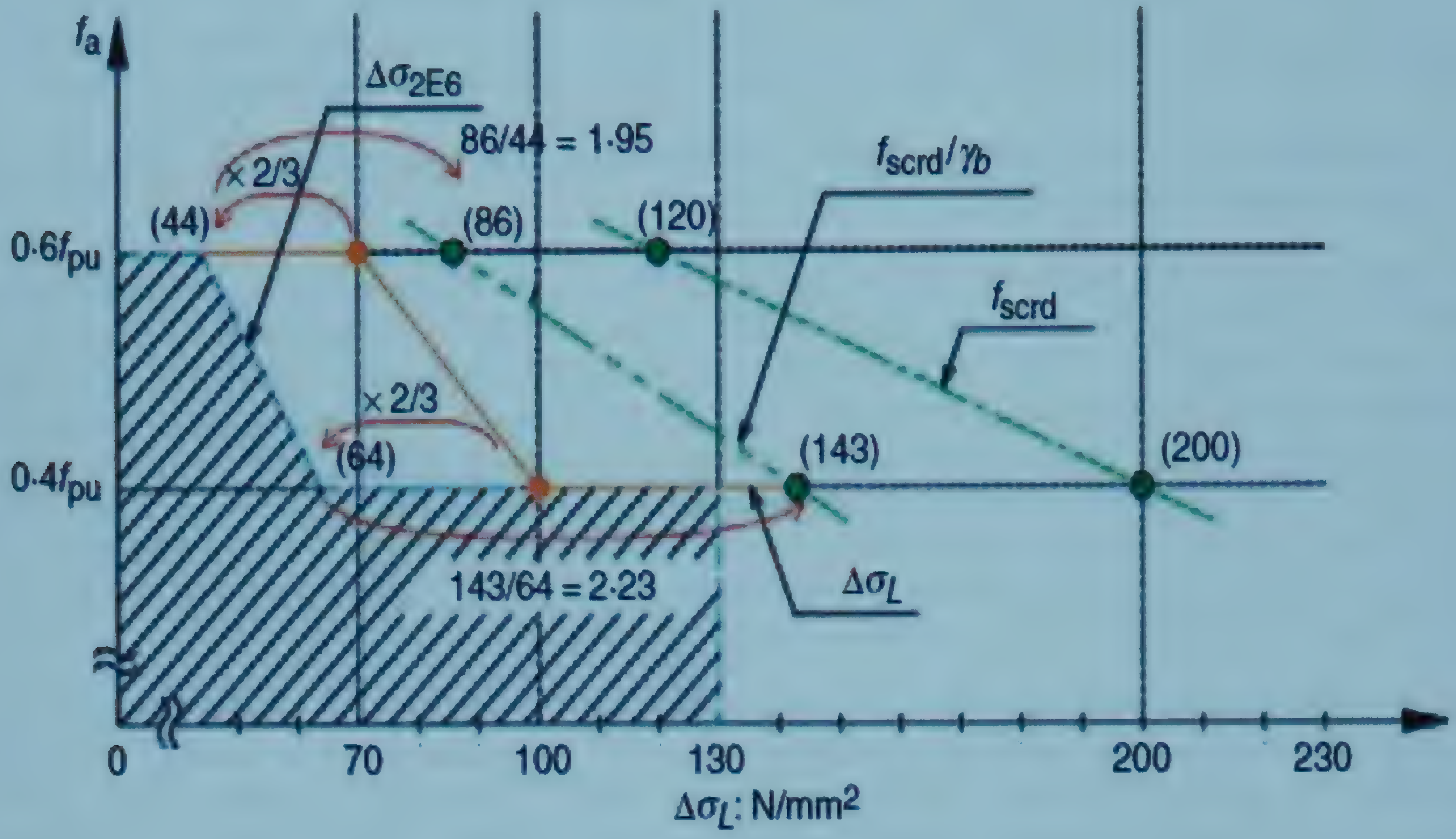


Fig. A2.1 : Comparison of Method A and Method B

According to this specification designer can choose the tensile stress in each stay cable from 0.4f_{pu} to 0.6f_{pu} **Fig. A2.2** continuously and not one value of tensile stress in one bridge.

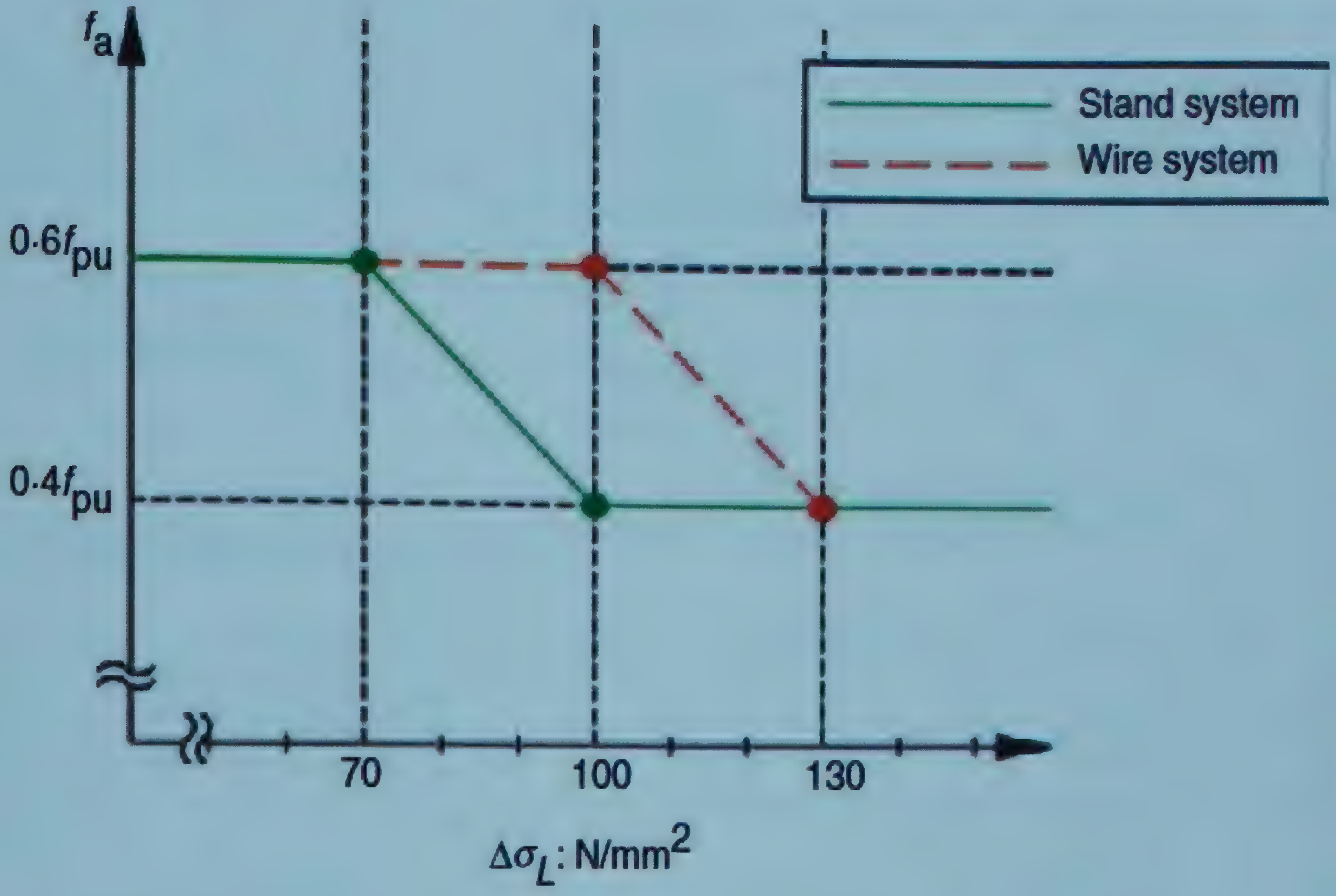


Fig. A2.2 Allowable stress in SLS to Design LL stress variation

Fig. A2.3 shows the design flow for stay-cable design.

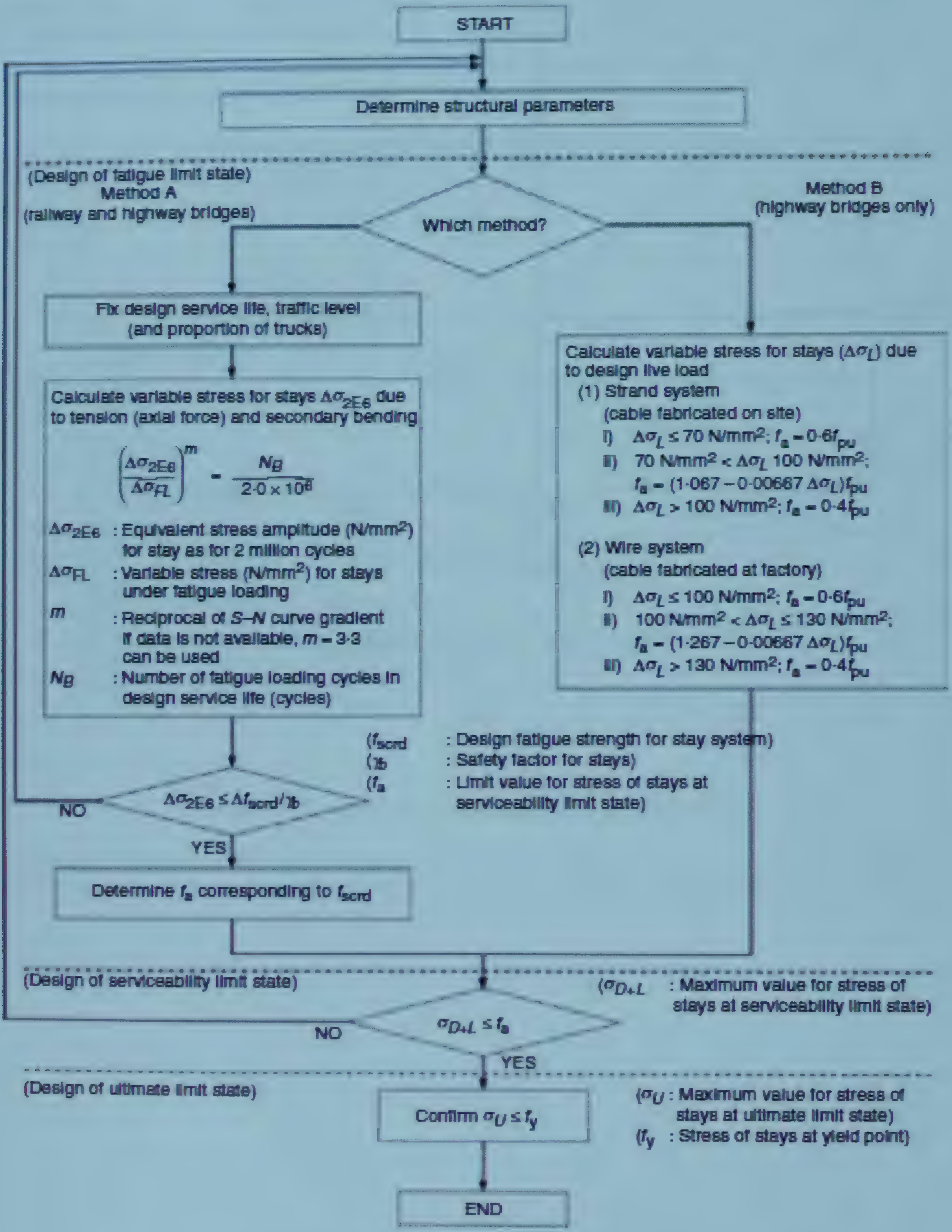

START
Determine structural parameters
(Design of fatigue limit state)
Method A
(railway and highway bridges)
Which method?
Method B
(highway bridges only)
Fix design service life, traffic level
(and proportion of trucks)
Calculate variable stress for stays $\Delta\sigma_{2E6}$ due to tension (axial force) and secondary bending
$\left(\frac{\Delta\sigma_{2E6}}{\Delta\sigma_{FL}}\right)^m = \frac{N_B}{2 \cdot 0 \times 10^6}$
$\Delta\sigma_{2E6}$: Equivalent stress amplitude (N/mm^2) for stay as for 2 million cycles
$\Delta\sigma_{FL}$: Variable stress (N/mm^2) for stays under fatigue loading
m : Reciprocal of S–N curve gradient If data is not available, $m = 3{\cdot}3$ can be used
N_B : Number of fatigue loading cycles in design service life (cycles)
Calculate variable stress for stays ($\Delta\sigma_L$) due to design live load
(1) Strand system
(cable fabricated on site)
i) $\Delta\sigma_L \le 70$ N/mm^2; $f_a = 0{\cdot}6 f_{pu}$
ii) 70 N/mm^2 < $\Delta\sigma_L$ 100 N/mm^2;
$f_a = (1{\cdot}067 - 0{\cdot}00667\,\Delta\sigma_L) f_{pu}$
iii) $\Delta\sigma_L > 100$ N/mm^2; $f_a = 0{\cdot}4 f_{pu}$
(2) Wire system
(cable fabricated at factory)
i) $\Delta\sigma_L \le 100$ N/mm^2; $f_a = 0{\cdot}6 f_{pu}$
ii) 100 N/mm^2 < $\Delta\sigma_L \le 130$ N/mm^2;
$f_a = (1{\cdot}267 - 0{\cdot}00667\,\Delta\sigma_L) f_{pu}$
iii) $\Delta\sigma_L > 130$ N/mm^2; $f_a = 0{\cdot}4 f_{pu}$
(f_{scrd} : Design fatigue strength for stay system)
(γ_b : Safety factor for stays)
(f_a : Limit value for stress of stays at serviceability limit state)
NO
$\Delta\sigma_{2E6} \le \Delta f_{scrd}/\gamma_b$
YES
Determine f_a corresponding to f_{scrd}
(Design of serviceability limit state)
(σ_{D+L} : Maximum value for stress of stays at serviceability limit state)
NO
$\sigma_{D+L} \le f_a$
YES
(Design of ultimate limit state)
Confirm $\sigma_U \le f_y$
(σ_U : Maximum value for stress of stays at ultimate limit state)
(f_y : Stress of stays at yield point)
END

Fig. A2.3 Flow Chart for Stay Cable design as per JPCI Specifications

II. Setra Recommendation - Cable Stays

The SETRA recommendations relate the allowable axial stress σa at SLS for the stay cables with the stress variation $\Delta\sigma_L$ due to SLS live loads and can be determined from the following equation. The allowable axial stress σ_a is applicable for the entire stay cable system [SETRA 2002] unlike JPCI recommendation which applies to individual cables.

$$\sigma_a = \left| \begin{array}{lll} 0.6\,\sigma_{UTS} & & \text{if } \Delta\sigma_L \le 50\,\text{MPa} \\ \sigma_a \le 0.46\left(\dfrac{\Delta\sigma_L}{140}\right)^{-0.25} & \sigma_{UTS} & \text{if } 50 < \Delta\sigma_L \le 140\ \text{MPa} \\ 0.46\,\sigma_{UTS} & & \text{if } \Delta\sigma_L > 140\,\text{MPa} \end{array} \right.$$

The SLS design method demonstrated above involves implicitly the fatigue consideration for the stay cable system.

The explicit fatigue limit state design requires the stay cable system to be tested for two million cycles for specific upper stress limits and specific amplitudes. For cable stayed bridges, the upper stress limit is taken usually as 45% f_{UTS}. The amplitude is commonly taken as 160 MPa for strand systems and 200 MPa for wire systems. Upon carrying out the fatigue test, appropriate safety factors is taken into account to specify the allowable stress variation.

R. Walter [Walter et al. 1999] suggests simplified method for the definition of the permissible stress variation $\Delta\sigma_{per}$ by establishing the Woehler curve **Fig. A2.4** from the available test results then deducing the allowable stress variation $\Delta\sigma_{per}$ by using the following equation:

$$\Delta\sigma_{per} = \frac{\Delta\sigma_{test}}{\gamma_{Mf}} \approx \frac{\Delta\sigma_{test}}{1.5}$$

Where:

γ_{Mf} =1.5 partial safety factor (as per SETRA)

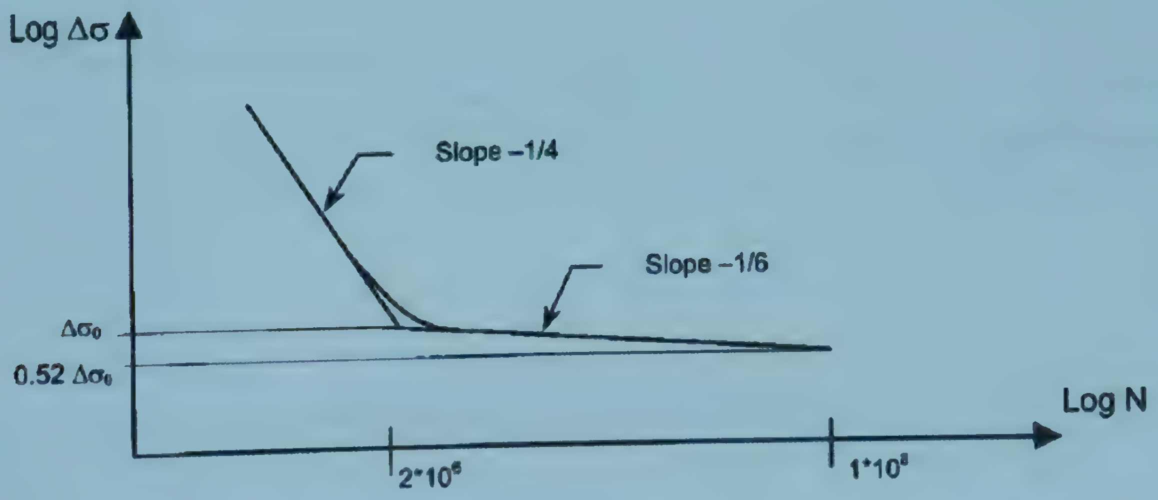


Fig. A2.4 Typical fatigue Curve for Stay Cable as per SETRA 2002

Upon the determination of $\Delta\sigma_{test}$, the stress variation due to the fatigue load should be calculated (using a structural computer model) and should be less than ($0.52\ \Delta\sigma_0 / 1.5$). For a stay cable system with $\Delta\sigma_0 = 200$ MPa for example, the allowable stress variation due to the fatigue load = ($0.52 \times 200/1.5$) ≈ 70 MPa. The fatigue load for road bridges for design shall be as per the Eurocode EN 1991-2-2003. The stress due to fatigue live load should be less than 70 MPa as per SETRA.

Based on the previous experience with the design of cable-stayed bridges with concrete decks, the stress variations due to fatigue loads like that of the Model 3 of Eurocode EN 1991-2-2003, are often far below 70 MPa. The SLS design governs therefore the required size of the stay cables. Similarly, for extradosed bridges, where the stress variations due to live loads are less than the corresponding stress variations for the cable-stayed bridges, the SLS design is expected to also govern the size of the stay cables and the permissible stress at SLS is increased as allowed for by SETRA recommendations (2002).

III. PTI Recommendations for Stay cable design, Testing and Installation

The philosophy adopted by PTI for performance of the cable, is to start with the single 7-wire element fatigue capacity of 33 ksi (229 MPa) **Fig. A2.5**, then reduce this by a value, Δ_1, to cover quality assurance considerations, and a value, Δ_2, to cover length effects, anchorage stress risers and factor of safety. The quality assurance value considers the difference from the performance of an individual strand element compared to a group of strands forming a cable. It includes interaction effects, cable. twist, and assembly variances, and is based on experience from load testing results of many assembled cables. For strand cables the Δ_2 correction used by PTI is 10 ksi (69 MPa) and the Δ_1 correction is 4.93 ksi (35 MPa). **Fig. A2.5** shows application of these corrections, resulting in an allowable fatigue stress range for a cable comprised of multiple 7-wire strands of 19 ksi (124 MPa) at 2 million cycles, based on a maximum stress of 45% of the ultimate strength of the strand

The PTI 3rd edition and earlier were based on an allowable stress design procedure where the cables were sized based on a maximum stress of 0.45 f's for Group I loading, then a check of the fatigue range of 18.1 ksi based on the selected cable size. If based on this check the fatigue limit is not met, then fatigue governs, and the cable size is increased to reduce the stress range.

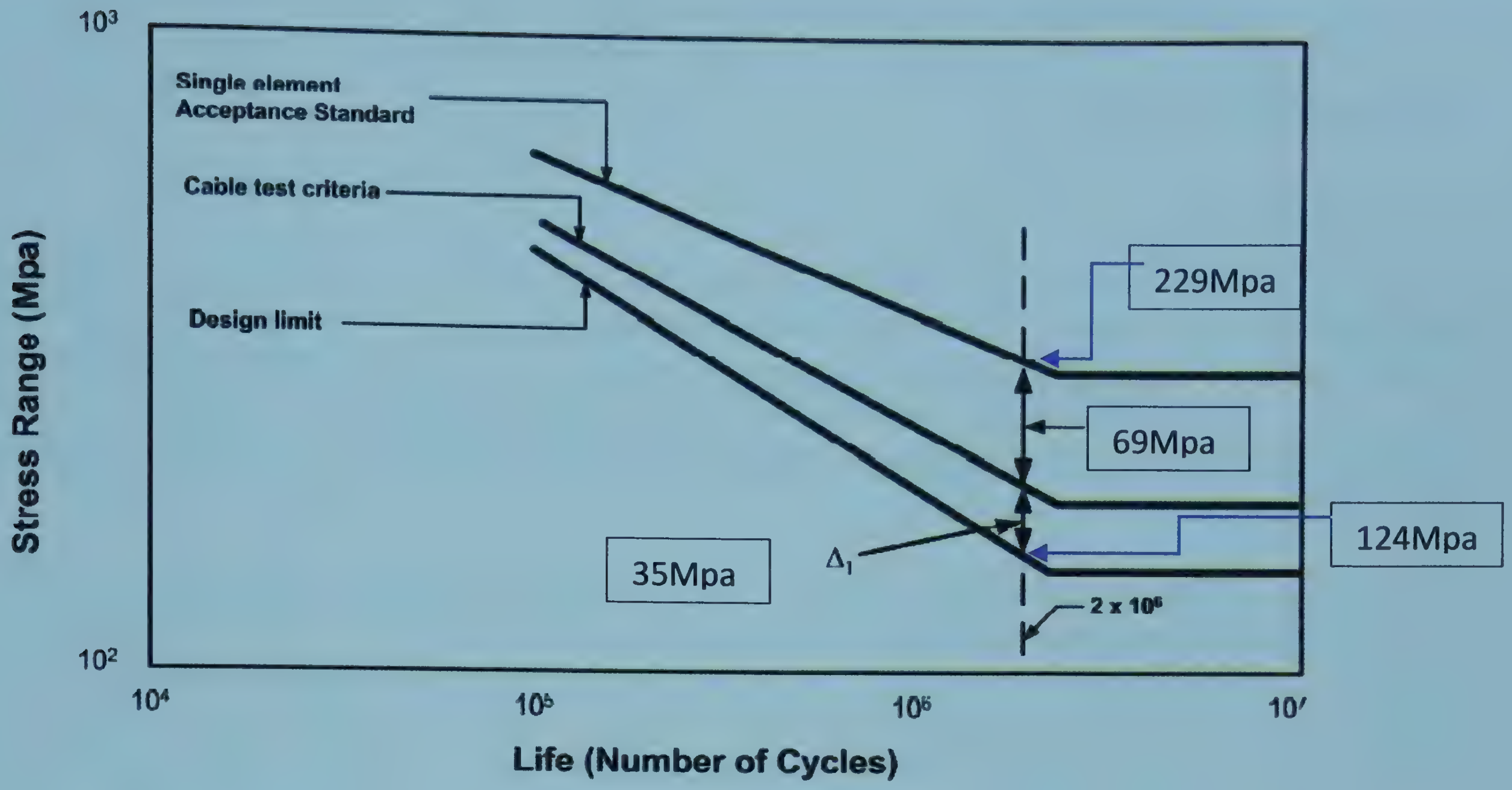


Fig. A2.5 Strength resistance Factor, (PTI DC45.1-18)

The current PTI specification is based on a Load Resistance Factor Design (LRFD) philosophy, meaning that the four basic limit states of service, strength, fatigue and extreme event must satisfy for the following expression:

$$\eta\sum\gamma_1 Q_i \leq \phi\, R_n = R_r$$

where :

γ_1	is the load factor
ϕ	is the resistance factor
Q_i	is the force effect, axial load, or axial plus bending, as applicable
R_n	is the nominal resistance.
R_r	is the factored resistance; and
$\eta >$	0.95 is a factor relating to ductility, redundancy, and operational importance. (Refer to Sections 1.3.3, 1.3.4, and 1.3.5 of AASHTO LRFD.). The factor η = 1 for these Recommendations.

For the limit states referred to in Table 3.4.1-1 of AASHTO LRFD, the following resistance factors shall apply:

Strength A – Axial Only ϕ = 0.65 to 0.75 per diagram in **Fig. A2.6**

Strength B – Axial and Bending combined ϕ = 0.78

Extreme Event ϕ = 0.95

Fatigue ϕ = 1.0

In PTI recommendations", 5th Edition an attempt to include term extradosed bridges in the specification was made. These provisions allowed design to an equivalent allowable stay stress of 0.6 f's for the very specific class of bridges where the fatigue stress range is less than 2.5%. While this went part way in recognizing extradosed bridges, it allowed no flexibility on intermediate conditions where the fatigue stress is more than 2.5% but less than a typical cable-stayed bridge.

PTI, 6th Ed. edition removed the specific references to extradosed bridges in the 5th edition, to include provisions that address bridges with relatively low live load demands on the stay cables in a more general manner. This is accomplished by allowing a transition in the stay capacity for low demand cables, expressing this as a function of the total live load plus wind stress to maximum ultimate tensile strength ratio (Total LL+W/MUTS) for the cable. The material factor are considered to recognize the lower live load fatigue demand on bridges that have relatively small live load demand on the cables. For the total LL+W/MUTS ratio over 7.5% the material factor is 0.65 as for a normal cable stayed bridge. For a Total LL+W/MUTS ratio of 2.5% a factor of 0.75 is allowed, essentially allowing a maximum stay stress of 0.6 f's. A linear transition is permitted between these two limits **Fig. A2.6**.

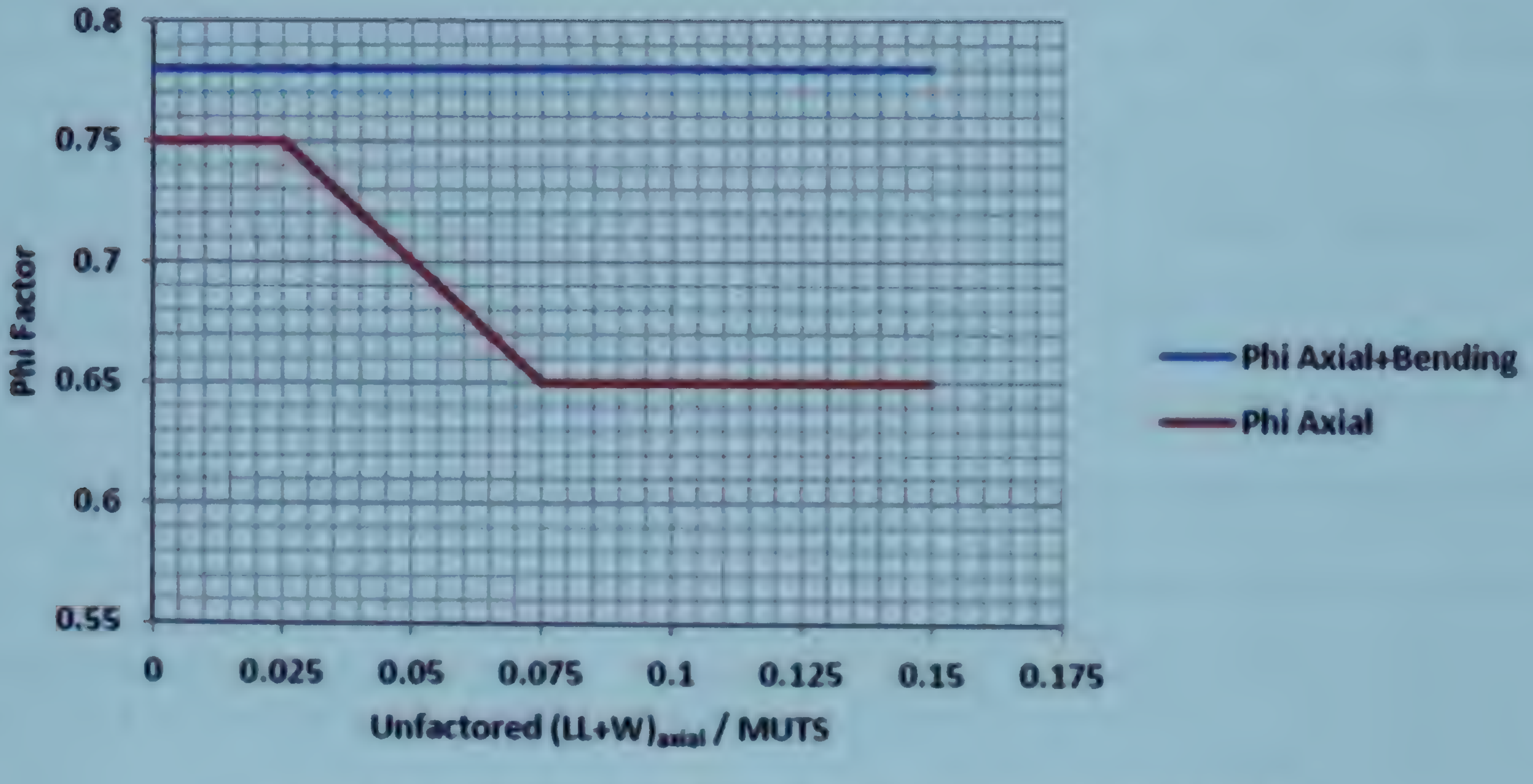


Fig. A2.6 Strength resistance Factor, (PTI DC45.1-18)

The PTI Recommendations requires that the cable forces comply with a Strength Limit State and a Fatigue Limit State. For a theoretically infinite fatigue life, the maximum stress range in each cable must be less than the constant amplitude fatigue threshold stress. The fatigue load consists of a single design truck, in a single lane, and the load effect is then increased by a Dynamic Load Allowance of 15% and by a factor of 1.4 to account for longer spans of cable-stayed bridges than those for which the fatigue provisions of the AASHTO

LRFD (AASHTO 2004) were originally designed. The factored load effect $\gamma(\Delta F)$ must be less than half the constant amplitude fatigue threshold $(\Delta F)_{TH}$ as described by the following equations.

$$\gamma(\Delta F) \leq (\Delta F)_n$$

$$\gamma(\Delta F) \leq \frac{1}{2}(\Delta F)_{TH}$$

where (ΔF) is the stress range due to the passage of the fatigue load

$(\Delta F)_{TH}$ is the constant amplitude fatigue threshold (taken as 110 MPa for parallel strands)

γ is the load factor of 0.75

$$\therefore 0.75\ (\Delta F) \leq \frac{1}{2}(110)$$

$$(\Delta F) \leq 73 \text{ MPa}$$

For AASHTO LRFD, stress range due to fatigue live load must be less than 73 MPa.

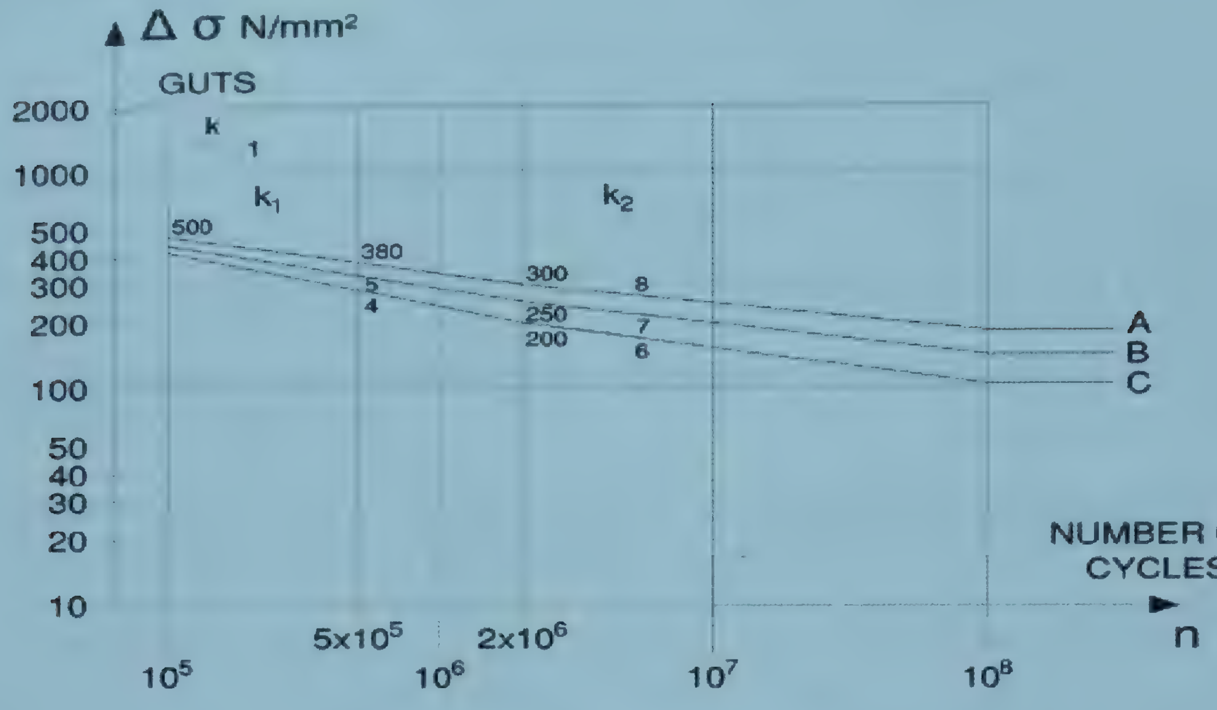
IV. Acceptance of Cable systems using prestressing steel – FIB 89

As per FIB 89, fatigue design should be carried out for stay cables by the method similar to SETRA as explained above with material factor of 1.25 and Wohler Curve C **Fig. A2.7** which represents minimum test performance requirements of stayed / Extradosed cables.

Also the permissible stresses in the stay cables system tested in accordance with FIB 89 **Chapter 6** shall be 50% GUTS for cable stay and for extradosed cable system shall be 60% GUTS.

STRAND - STAY CABLE

STRESS RANGE



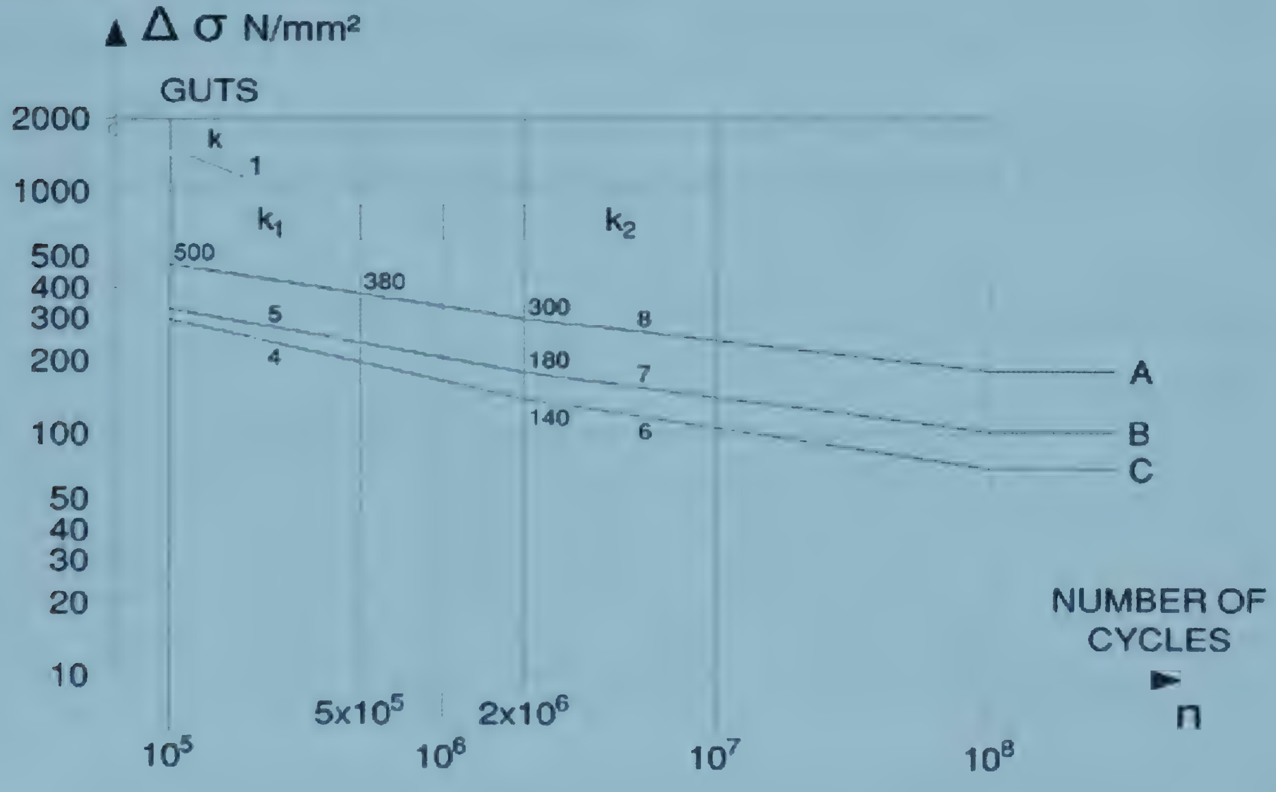
A = SINGLE TENSILE ELEMENT, UPPER STRESS 45% AUTS

B = ANCHORED SINGLE TENSILE ELEMENT, UPPER STRESS 45% GUTS

C = CABLE TEST, UPPER STRESS 45% GUTS

STRAND - EXTRADOSED CABLE

STRESS RANGE



A = SINGLE TENSILE ELEMENT, UPPER STRESS 45% AUTS

B = ANCHORED SINGLE TENSILE ELEMENT, UPPER STRESS 55% GUTS

C = CABLE TEST, UPPER STRESS 55% GUTS

Fig. A2.7 S-N Curve for cable system as per FIB 89

Annexure A3
(Pertaining to Chapter 9)

Additional Information on Wind Tunnel Testing

This Annexure provides more information about wind tunnel testing of bridge using (i) section model (ii) Full bridge model

Tests on section models of bridge decks can be used to determine the mean or static components of the overall wind load on the model. These wind loads can be obtained using rigid models with geometrically scaled features. The effect of wind inclination in elevation should be examined. The extent of the effect should be judged depending on the site topography, any planned super-elevation of the bridge and predicted torsional deflections under traffic loads. Generally, tests up to ± 6° are adequate. Accurate measurements of both the mean and the dynamic components of the overall loads can only be obtained if both the approach flow and the local environment are properly simulated. For the scale of model bridge required, this becomes impractical.

The other important objective of section model tests is to determine the aerodynamic stability of the bridge deck, mounted with deck furniture, using a geometrically scaled model of a section of the bridge elastically mounted in a wind tunnel. Typically, such models simulate the lowest bending and torsional vibration frequencies, and are tested in uniform laminar flow. This is expected to yield conservative results, as compared to those obtained in turbulent flow. In more advanced or refined stages, section models are tested in simulated turbulent flow in order to provide estimates of the responses at sub-critical wind speeds. As the simulated turbulence generally has a preponderance of the smaller-size eddies most likely to influence flow features such as vortex-shedding or re-attachment, the total intensity of turbulence should be selected with care. Generally, this should be significantly lower than the standard atmospheric value for full scale. Reliance on beneficial effects from turbulence should not be allowed to reduce the likely aerodynamic effects. In addition to modelling the geometry, it is necessary to maintain a correct scaling of inertia forces, the time scale, the frequency, and the structural damping. The time scale is normally set indirectly by maintaining the equality of the model and full-scale reduced velocities of particular modes of vibration. The reduced velocity is the ratio of a reference wind speed and the product of a characteristic length and the relevant frequency of vibration.

Measurements should be carried out through the range of wind speeds likely to occur at the site to provide information on both relatively common events, influencing serviceability, and relatively rare events, which govern ultimate strength behaviour. Wind inclination in elevation should be examined.

Measurements of vortex excitation require careful control of the wind speed around the critical velocity, and any predicted divergent amplitudes should be assessed to ensure they do not become so violent as to destroy the model.

A Sectional model test can furnish most of the relevant information required for the aerodynamic design of the bridge. One limitation in modelling herein is that the Reynolds number can not be simulated. However, fortunately this distortion has insignificant effect because of the deck section having sharp edges. Furthermore, it is to be remembered that details at the edge of the section can have a considerable effect on its aerodynamic response. Thus, attachments such as, fairings, guide vanes and deflectors are considered for this purpose.

A dynamic model of the full bridge, commonly referred to as an aero-elastic model, can be used in the wind tunnel, to provide information on the overall wind induced mean and/or dynamic loads and responses of bridges. Such models are particularly valuable for slender, flexible and dynamically sensitive structures, where dynamic response effects may be significant. To be representative the tests should consistently model the salient characteristics of natural wind at the site and the aerodynamically significant features of the geometry of the bridge. It is also necessary to correctly model the stiffness, mass and damping properties of the structural system.

It is only possible to model the full spectrum of atmospheric turbulence in a wind tunnel at small scale; together with the obvious constraint of fitting a full bridge model within the tunnel, this is generally irreconcilable with the scale desirable to ensure correct behaviour, which is commonly sensitive to small changes in cross-section. A full model should therefore be used only where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations necessitate its use. Particular care is needed in its design and interpretation of the test results. It is noteworthy that modern techniques of model fabrication and very highly sophisticated instruments such as accelerometers, anemometers, and pressure transducers/scanners make it possible to monitor model responses very accurately.

ANNEXURE A4
(Pertaining to Chapter 10)

A4.1. APPROACHES OF INTERNATIONAL GUIDELINES FOR TESTING OF STAY CABLE

Table A4.1

INTERNATIONAL CODES	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7th EDITION) [3]
1 CABLE STAY BRIDGES	✓	✓	✓
2 EXTRADOSED BRIDGES	✓	✓	
UPDATED	First print published in 2002 & the same will not be updated further.	Latest Version published in March- 2019	Latest Version published in Nov- 2018

Notes:-

1 If for a particular project, all the cables have $\Delta\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) is less than 50 MPa then, qualification testing (initial as well as suitability testing) requirements of the extradosed bridges shall be adopted else all the testing requirements of the cable stayed bridges shall be adopted.

2 Apart from initial & suitability testing, other material checks on strands, PE sheathed duct, filler material and HDPE duct, wedges and anchorages parts shall be also carried out **[2]**.

A4.2. COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE FOR CABLE STAYED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF CABLE STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF CABLE STAY- SADDLE ASSEMBLY ✓

Table A4.2

	Test Detail	**SETRA** [1]	**fib-89 (CEB-FIP)** [2]	**PTI (7th EDITION)** [3]	**Recommended Code For Testing**
S.No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
2	ANCHORAGE /SADDLE BENDING FATIGUE & TENSILE TESTING		✓✓		**fib-89**
3	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓	✓✓	**fib-89**
4	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓	✓	**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
3	ANCHORAGE BENDING FATIGUE & TENSILE TESTING	NOT REQUIRED TO BE PERFORMED			
4	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
5	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
6	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**

A4.3 COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE TESTING FOR EXTRADOSED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF EXTRADOSED STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF EXTRADOSED STAY- SADDLE ASSEMBLY ✓

Table A4.3

	Test Detail	**SETRA** [1]	**fib-89 (CEB-FIP)** [2]	**PTI (7th EDITION)** [3]	**Recommended Code For Testing**
S.No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
2	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓		**fib-89**
3	SADDLE FRICTION TESTING	✓	✓		**fib-89**
4	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓		**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
3	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
4	SADDLE FRICTION TESTING	✓	✓		**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**

A4.4. Applicable Property, Acceptance Criteria & Test Method & Frequency of Testing of Materials

Table A4.4.1 Recommended Values for Prestressing Steels for Cables

Type of steel	E-modulus	Diameter	Tensile strength	Cross section	Recommended tolerances on mass	Characteristic ultimate force
	GPa	mm	MPa	mm^2	%	kN
Strands	195 ± 5%	15.2	1860	140	± 2	260.4
		15.7	1770	150	± 2	265.5
			1860			279.0
Wires	205 ± 5%	6	1770	28.27	± 2	50.1
		7	1670	38.5		64.3
		7	1770	38.5	± 2	68.1

Table A4.4.2 Mechanical Characteristics for Prestressing Steels

Property			**Acceptance Criteria**	**Test Method**
Elongation at maximum force			Not less than 3.5%	ISO 15630-3[S7]
Constriction at break			Ductile break visible to naked eye, constriction coefficient ≥25%	ISO 15630-3
Relaxation 1000h at 0.70 AUTS, 20°C			Not more than 2.5%1)	ISO 15630-3
Fatigue stress range with an upper stress limit of 0.45 AUTS	Strands	300 MPa	At least 2x10^{6} cycles	ISO 15630-3
	Wires	370 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 2 x 10^{6} cycles)	Strands	380 MPa	At least 5 x10^{5} cycles	ISO 15630-3
	Wires	465 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 5 x 10^{5} cycles)	Strands	500 MPa	At least 1 x10^{5} cycles	ISO 15630-3
	Wires	610 MPa		
Tensile test after the fatigue test			95% of AUTS	ISO 15630-3
Deflected tensile test of Strands			Not more than 20%	ISO 15630-3

1)The values are applicable for testing of samples taken just after manufacturing.

Above specified properties apply to hot dipped metallically coated or epoxy coated prestressing steel. However, they also have to be satisfied after application of the filling material and sheathing.

Table A4.4.3 Requirements for metallic coatings on prestressing steel[S12]

Property		Acceptance Criteria	Test Method
Zinc or zinc/aluminium coating	-Continuity	No defects	EN 10244-1 and 2[S13]
	-Weight of coating	190 to 350 g/m^2	
	-Adherence	No defects, cracks	
	-Appearance	Smooth, no zinc drop	

Table A4.4.4 Requirements for epoxy coatings on prestressing steel

Property	Acceptance Criteria	Test Method
Coating thickness	380 to 1140 µm	ASTM A 882/A882M[S14]
Salt spray test	No corrosion, holidays[1)] nor other coating damage present after 3,000 hours exposure to salt spray (fog)@ 70% GUTS	ASTM B117[S4]
Chloride permeability	No chloride penetration detected	FHWA-RD-74-18
Chemical resistance	2 types of samples tested: Normal epoxy coated strand and sample with 1/4" hole intentionally drilled into coating. No coating damage after 45 days immersion. Steel corrosion at drilled hole did not cause adjacent epoxy coating to soften, blister, nor lose bond with adjacent steel.	ASTM G 20[S49]
Impact test	No cracking, shattering or bond loss	ASTM G 14[S23]
Sand abrasion test	< 10 mils (0.25 mm) loss of coating	ASTM D 968[S50]
Bending test	No cracking or debonding when bending strand around mandrel 32 times nominal strand diameter	ASTM A 370[S51]
Adhesion of coating	No coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position	ASTM A 882/A882M[2)]

[1)] Holiday detection: Dry-Type continuous in line detector with minimum voltage of 3 KV would be applied.

[2)] No cracks visible to the unaided eye shall occur in the coating upto an elongation of 1% , or no coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position.

Table A4.4.5 Grease Specification

Property		Acceptance Criteria	Test Method
Cone penetration,60 strokes at 25^0 C (1/10 mm)		220 – 300	ISO 2137[S24] or ASTM D217[S25]
Dropping point		≥ 150^0 C	ISO 2176[S26] or ASTM D566[S27]
Heat resistance, flash point		≥ 250^0 C	EN ISO 2592[S40]
Oil separation	At 40^0C	At 72 hours : ≤ 2.5% At 7 days : ≤ 4.5%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
	At 100^0C	At 50 hours : ≤ 4%	
Oxidation stability		100 hours at 100^0C: ≤ 0.06 MPa 1000 hours at 100^0C: ≤ 0.2 MPa	DIN 51 808[S32] Or ASTM D942[S33]
Corrosion Test[1]	168 hours at 35^0C	No corrosion	EN ISO 9227 (NSS test)[1],[S34]
	168 hours at 35^0C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)
Rust prevention		Grade : 0	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-, S^{2-}, NO^-_3	≤ 50 ppm (0.005%)	NFM 07-023[S37],[2]
	SO_4^{2-}	≤ 100 ppm (0.010%)	NFM 07-023[2]

[1] Test sample consists of a structural steel plate S355 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of grease of a maximum thickness of 125 μm. This thickness can be controlled with eddy current or by measuring the mass.

[2] Applied accordingly to grease. Requirements for grease pertaining to fluidity, de-oiling , water absorption, saponification, micro-biological resistance, coefficient of expansion must be satisfied or declared.

Table A4.4.6 Wax Specification

Property	Acceptance Criteria	Test Method
Congealing point	≥ 65^0C	ISO 2207[S38]
Heat resistance, dropping point	≥ 60^0C	ISO 2176[S39]
Heat resistance, flash point	≥ 250^0C	ISO 2592[S40]
Penetration (1/10 mm) at 25^0C	≤ 125 (in 1/10 mm)	ISO 2137[S41]
Cold resistance	No cracking at -40^0C	ISO 2137[S41]

Bleeding at 40^0C		≤1.0%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
Resistance to oxidation 100 hours at 100^0C		≤ 0.03 MPa	ASTM D942[S33]
Copper-strip corrosion 100 hours at 100^0C		Class 1a	ISO 2160[S42]
Corrosion Test[1]	168 hours at 35^0C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)[S34]
	168 hours at 35^0C	No corrosion	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-,S^{2-}, NO_3^-:	≤ 50 ppm (0.005%)	NFM 07-023[S37]
	SO_4^{2-} :	≤ 100 ppm (0.010%)	NFM 07-023

[1] Test sample consists of a structural steel plate Fe 510 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of wax of a maximum thickness of 125 µm. This thickness can be controlled with eddy current or by measuring the mass.

Table A4.4.7 Characteristics of PE sheathing

Property	**Acceptance Criteria**	**Test Method**
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black	Index is max. C 2	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on sheathing	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 250% on sheathing	ISO 527-2
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on sheathing	ISO 527-2
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO11357-6[S20]

Table A4.4.8 Characteristics of the sheathed cable strands

Property	**Acceptance Criteria**	**Test Method**
Sheathing thickness	≥ 1.5 mm	Calibrated gauge
Quantity of filling material	≥ ± 20% of specified value	Calibrated gauge
Friction resistance (specimen length of 0.3)	≥ 667 N	XP A 35-037.1[S5]
Leak tightness / migration[1)]	Increase mass due to absorption Of water shall be less than 0.8 g.	XP A 35-037.1 Static test
Impact resistance	No perforation of the sheathing	XP A 35-037.1, 3 ASTM G14[S23]

1)The value are applicable for testing of samples taken just after manufacturing.

Table A4.4.9 Characteristics of HDPE cable pipe

Property	**Acceptance Criteria**	**Test Method**
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black[1)]	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black[1)]	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black[1)]	Index is max. C 2w	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on pipe	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 350% on pipe	ISO 527-2 ISO 6259-3[S46]
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on pipe	ISO 527-2, ISO 6259-3
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO 11357-6[S20]
Thermal coefficient of dilation	Value to be declared by manufacturer	ISO11359-2[S22]
Bending modulus	≥ 750 MPa at 23 ^{0}C on raw material ≥ 600 MPa at 23 ^{0}C on pipe	ISO 178[S45]
Environmental stress cracking resistance	Test with min of 600 hours Cond C F20	ISO 22088[S55] or ASTM D1693[S44]

A4.5. Quality control testing of cable systems

Property	**Test Frequency**
Prestressing steel according to Table A4.4.2 Mechanical characteristics for prestressing steels	
Tensile strength with elongation at maximum force	1 test per strand coil[1)]
Constriction at break	
Relaxation	1 test par batch of prestressing steel[2)]
Fatigue strength with subsequent tensile test	2 tests every 100 tonnes of strand[3)]
Deflected tensile test	1 test every 100 tonnes of strand[3)]
Metallic coating according to Table A4.4.3 Requirements for metallic coating on prestressing steel^{S12}	
Continuity	1 test every 20 tonnes of strand[3)]
Weight of coating	
Adherence	
Epoxy coating according to Table A4.4.4 Requirements for epoxy coating on prestressing steel	
Coating thickness	1 test per strand coil[1)]
Salt spray test	Only for initial type testing, not applicable for factory production control
Chloride permeability	
Chemical resistance	
Impact test	
Sand abrasion test	
Bending Test	1 test per strand coil[1)]
Adhesion of coating	1 test per strand coil[1)]
Relaxation	1 test per batch of prestressing steel[2)]
Grease filling material according to Table A4.4.5 Grease specification	
Cone penetration	1 test per batch[5)] of grease
Dropping point	
Heat resistance, flash point	
Oil separation at 40°C	
Oil separation at 100°C	
Oxidation stability	Only for initial type testing, not applicable for factory production control
Corrosion test (NSS test)	
Corrosion test (AHT test)	

Rust prevention	1 test per batch5) of grease
Content of aggressive elements	
Wax filling material according to Table A4.4.6 Wax specification	
Congealing point	1 test per batch5) of wax
Heat resistance, dropping point	
Heat resistance flash point	
Penetration at 25°C	
Cold resistance	
Bleeding at 40°C	
Resistance to oxidation 100h at 100°C	Only for initial type testing, not applicable for factory production control
Copper-strip corrosion 100h at 1 00°C	
Corrosion test (NSS test)	
Corrosion test (AHT test)	
Content of aggressive elements	1 test per batch5) of wax
Sheathing according to Table A4.4.7 Characteristics of PE sheathing	
Melt index	1 test per batch of raw material4)
Specific weight, Density	
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	1 test per batch of raw material4) 3 tests every 10 tonnes of sheathing material
Elongation at break at 23^{0}C	
Elongation at break at -20^{0}C	
Thermal stability under O^2	1 test per batch of raw material4)
Completed strand according to Table A4.4.8 Characteristics of the sheathed cable strands	
Sheathing thickness	1 test per strand coil1)
Quantity of filling material	1 test per strand coil1)
Friction resistance	3 tests every 100 tonnes of strand3)
Leak tightness	
Impact resistance	
HDPE cable pipe according to Table A4.4.9 Characteristics of HDPE cable pipe	
Melt index	1 test per batch of raw material (on raw material4) 1 test per size of cable pipe but at least one test

	per raw material batch (on pipe)
Specific weight, Density	1 test per batch of raw material4)
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	3 tests per batch of raw material (on raw material4) 3 test per size of cable pipe in the project (on pipe)
Elongation at break at 23^{0}C	
Elongation at break at -20^{0}C	
Thermal stability under O^2	1 test per size of cable pipe but at least one test per raw material batch (on pipe)
Thermal coefficient of dilatation	Only for initial type testing, not applicable for factory production control
Bending modulus	3 tests per size of cable pipe but at least one test per raw material batch (on pipe)
Environmental Stress Cracking Resistance	Only for initial type testing, not applicable for factory production control

1) A coil is a delivery unit, weighing not more than 5 tonnes.

2) Batch corresponding to one heat number, weighing not more than 200 tonnes. Relaxation tests with minimum 120 h according to ISO 15630-3 and subsequent extrapolated relaxation values to 1000 h might be accepted.

3) Weight of bare steel

4) Batch corresponding to one production unit capacity, weighing not more than 30 tonnes

5) Quantity of product generally manufactured in one operation.

A4.6 References of Testing Standards

[S4] - ASTM B117-16 “Standard practice for operating salt spray (fog) apparatus”

[S5]- XP A35-037:2003 “Torons en acier a haute resistance proteges gaines”

[S7]- ISO 15630:2011 “Steel for reinforcement and prestressing of concrete”

[S8]- prEN 10138:2009 “Prestressing Steel”

[S13]- EN 10244:2009 “Steel wire and wire products, non-ferrous metallic coatings on steel wire”

[S14]- ASTM A 882 M-10 “Standard specification for filled Epoxy -coated Seven Wire Prestressing steel strand”

[S16]- ISO 1133:2011 “Plastics-Determination of the melt flow rate (MFR) and the melt volume rate (MVR) of thermoplastics”

[S17]- ISO 1183:2012 “Plastics-Methods for determining the density of non-cellular plastics”

[S18]- ISO 6964:2006 “Polyolefin pipes and fittings-Determination of carbon black content by calcinations and pyrolysis-Test method and base specification”

[S19]- ISO 18553:2002 “Method for the assessment of the degree of pigment or carbon black dispersion in Polyolefin pipes, fittings and compound

[S20]- ISO 527:2012 “Plastics- Determination of tensile properties”

[S22]- ISO 11359-2:1999 “Plastics- Thermo mechanical analysis (TMA)-Part-2: Determination of coefficient pf linear thermal expansion and glass transition temperature”

[S23]- ASTM G14-04 “Standard test method for impact resistance of pipeline coatings (Falling Weight Test)”

[S24]- ISO 2137:2007 “Petroleum products -Lubricating grease and petroleum - Determination of cone penetration”

[S25]- ASTM D217 -16 “Standard test methods for cone penetration of lubricating Grease”

[S26]- ISO 2176:1995 “Petroleum products -Lubricating grease -Determination of dropping point”

[S28]- BS 2000-121:2005 “Method of test for petroleum and its products. Determination of Oil Separation from lubricating grease, Pressure filtration method”

[S29]- DIN 51817:2014 “Determination of Oil Separation from lubricating grease under static conditions”

[S30]- NFT 60 191:2011 “Petroleum products and Lubricating grease- Oil Separation On storage of Lubricating Grease- Method under pressure -Static conditions”

[S31]- ASTM D6184-16 “Standard Test Method for oil separation from Lubricating grease (Conical Sieve Method)”

[S32]- DIN 51808:2015 “Testing of Lubricants; determination of oxidation stability of greases: oxygen method”

[S33]- ASTM D942-15 “Standard Test method for Oxidation Stability of Lubricating Greases by the Oxygen Pressure Vessel Method”

[S34]- ISO 9227:2017 “Corrosion tests in artificial atmospheres-Salt spray tests”

[S35]- ISO 6270-2:2005 “Paints and varnishes-Determination of resistance to humidity - Part-2 Procedure for exposing test specimens in condensation-water atmospheres”

[S36]- ISO 11007:1997 “Petroleum products and Lubricants- Determination of rust prevention characteristics of lubricating greases”

[S37]- NFM 07-023:1969 “Liquid Fuel-Determination of Chlorides in crude petroleum and petroleum products”

[S38]- ISO 2207:1980 “Petroleum waxes-Determination of Congealing point”

[S39]- ISO 2176:1995 “Petroleum products-Lubricant greases-Determination of dropping point”

[S40]- ISO 2592:2000 “Determination of Flash and fire points – Cleveland open cup method”

[S41]- ISO 2137:2007 “Petroleum products and Lubricant - Determination of Cone penetration of lubricating greases and petrolatum”

[S42]- ISO 2160:1998 “Petroleum products- Corrosiveness to copper -Copper strip test”

[S44]- ASTM D1693 “Standard test method for environmental stress cracking of ethylene plastics”

[S45]- ISO 178:2010 “Plastic, Determination of bending modulus”

[S46]- ISO 6295-3:2015 “Thermoplastic pipes-Determination of testing properties-Part 3 - Polyolefin pipes”

[S49]- ASTM G20-10 “Standard Test Method for Chemical resistance of pipeline coatings”

[S50]- ASTM D968-16 “Standard Test Methods for Abrasion Resistance of Organic Coatings”

[S51]- ASTM A370-17 “Standard Test Methods and definitions for mechanical Testing of Steel Products”

[S55]- ISO 22088:2006 “Plastics-Determination of resistance to environmental stress cracking (ECS)”

Annexure A5

(Pertaining to Chapter 13)

Organisation Setup and Training Requirements for Inspection and Maintenance

This **Annexure** discuss about the minimum organizational setup and training requirements of personnel involved in operation and maintenance. This **Annexure** has been referred in **Chapter 13**.

A 5.1 The Organisational Set up depends on type of contract (IPC/HAM/BOT) and the agency responsible for carrying and the O&M. What follows is a general discussion as the agency may be AI, IE or the Owner- client

For the inspection and maintenance of an Cable Stayed bridge project a suitable organization set-up consisting of about 10 to 15 members headed by the Chief Engineer (Maintenance) or of equivalent rank in an O&M Agency outlining the responsibility of each & every staff (see **Table A.5.1**) shall be framed in such a manner that each element of the project shall be checked /inspected properly. The service level of each element of the project shall be assured for safe and undisturbed traffic movement.

Table A 5.1 Organisation Setup for inspection and Maintenance

Chief Engineer (Maintenance)		
Executive Engineer (Maintenance) for Bridges	Executive Engineer (Roads)	Executive Engineer (Maintenance) for Electrical components
Maintenance Engineer (Main EDB/ Approach spans	Maintenance Engineer (Roads)	Maintenance Engineer (Electrical)
Inspector (Bridges	Inspector (Roads)	Inspector (Electrical)

The Chief Engineer (Maintenance), the Executive Engineer (Maintenance), Maintenance Engineer for Bridge and Roads should be qualified civil engineers with at least 20, 15 and 10 years of working experience respectively, major part of which should be in inspection and maintenance of long span bridge structures.

The proposed organization set-up as in **Table A 5.1** may, however, be changed to suit the requirement of maintenance activities at site. The above organization set-up shall be augmented by Supervisors, System-in -Charge, Electrical, Secretary to the Chief Engineer (Maintenance), CAD draftsman, Enumerator, Computer operator and other office assistants as and when required for preparation of drawings, record keeping, estimating and help in other miscellaneous items.

It is essential that the inspection and maintenance team familiarize themselves thoroughly with design, construction and maintenance of these type of bridges.

Responsibilities

The duties of each inspection and maintenance officers as well assistants are mentioned below.

Chief Engineer (Maintenance) (one no.)

- Organize fortnight meeting to review the maintenance activities.
- Approval of the maintenance activities
- Allotment of fund
- Monitoring the maintenance works.
- Coordination with various agencies like traffic police, local administration, publicity media etc., in case of emergent repairs interruption to traffic by road blockage, etc.
- Monitoring the all-round safety

Duties of Executive Engineer (Maintenance) (one each for bridge, road and Electrical Components if any)

- Conducting special inspection, surveys and site investigations
- Finalization of inspection report
- Checking the cause of defects, structural inadequacy
- Finalization of maintenance scheme & estimates after reporting observations with suggestion for remedial action to Chief Maintenance Manager
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.
- Maintenance of lighting, lightening conductor and solar energy system, If any
- Supervision of the execution of the maintenance works
- Monitoring & approval of all safety aspects
- Necessary approval of inspection & maintenance works
- Coordination with various agencies like traffic police, local administration specialised execution agencies etc., in case of emergent repairs interruption to traffic by road blockage, etc.
- Finalizing the execution report.
- Preparation of the monitoring report

Duties of Maintenance Engineer (one each for bridge, road, electrical components, ROB)

- Conducting detailed inspection, surveys and site investigations as per approved format
- Prepare inspection report
- Analyse the cause of defects, structural inadequacy
- Prepare maintenance scheme & estimates after Reporting observations with suggestion for remedial action to Executive Maintenance Manager
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.
- Supervision of the execution of the maintenance works
- Preparation of the execution report.
- Arranging men, materials and machinery in advance as per requirement

Duties of Inspector

- Arranging men, materials and machinery in advance as per requirement
- Conducting detailed inspection, surveys and site investigations as per approved format
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency to the Maintenance Manager.
- Assist the execution of maintenance works

Duties of Supervisor

- Arranging men, materials and machinery in advance as per requirement
- Supervise the execution of maintenance works
- Assist detailed inspection, surveys and site investigations as per approved format
- Reporting to the Maintenance Manager.

Duties of System In-charge

- Routine inspection of the whole system

- Quick action to be taken in case of any type of fault
- Study the condition of whole system
- Supervise the execution of maintenance works
- Reporting to the Maintenance Manager

Duties of Electrician

- Daily inspection of each electrical unit
- Quick action to be taken in case of any type of fault
- Supervise the execution of maintenance works
- Reporting to the Maintenance Manager & System in-charge

A 5.2 Training

Training of all the personnel involved in maintenance is an integral part of the maintenance function. Such training, as may be required by the operating and maintenance personnel to achieve better performance can be classified as below:

- Training of Managers, Inspector and Supervisor.
- Training of Assistant, Electrician and gang men/mates.

(a) **Training of Managers, Inspector and Supervisor:** These officers are either diploma holders or fresh degree holders. Either way they are expected to perform all the necessary management tasks sometime without any formal management orientation or training. The typical section officer learns about maintenance management while he is on the job. Frequently the new officer handles the job in the same manner as the person before him. Techniques which may or may not have worked twenty years ago are tried again today. Manager and other officers must be provided with the opportunity to attend maintenance management and operations training. The areas to be covered during the training should be

- Identification of defects
- Planning maintenance operations effectively
- Crew scheduling and control to achieve higher productivity at low cost consistent with quality
- Problems of crew supervision—communicating and coordinating
- Preparing road inventory
- Management by objectives
- Repair methods

(b) **Training of Assistant, Electrician and gang men/mates:** The training of maintenance Assistant, Electrician and gang men/mates is very essential in two areas like (i) Safety aspects and (ii) Repair methods.

The training in safety aspects will include:

- Training of flag-men
- Use and lay out of temporary traffic signs for repair
- Safety oriented handling and parking of machinery and equipment.

The training in repair methods will include:

- Proper patching methods
- Proper grading of shoulders
- Maintenance of drainage structure
- crack sealing
- Painting and maintenance of the traffic signs and warnings
- Equipment maintenance.

Annexure A6

(Pertaining to Chapter 13)

Checklist for Inspection of Stay Cable System (Referred in Chapter 13)

A. Details about the Bridge and Stay Cable System

1) Bridge name and location
2) Year built
3) Span arrangement
4) Type of stay cable system used? See Clause 10.2 of these Guidelines
5) Name of Cable System Supplier
6) Type of anchorage used? Max and Min Number of stands for stay and max and min length of stay
7) Do cables go over "saddles" on the tower?
8) Type of cable outer barrier/stay pipe used
9) Do the cable systems have viscous dampers installed at deck levels?
10) Do cables have other dampers (other than viscous or neoprene rings) to control vibrations?
11) Do the cables have cross cables installed between them control vibrations?
12) Whether Coating/treatment has been provided on strands within free length of cable to enhance corrosion resistance? No/Yes
13) If Yes, Type of coating/treatment-
14) Are the coatings/treatments on cables discontinued or removed within the anchorage zone? yes, no, not known, not applicable)
15) Whether any of grout wax, grease has been used? Yes/No
16) If yes, type of grout etc.
17) Whether any filler material has been used in the anchorage zone- Yes/No
18) If yes, type of filler material used-
19) Have the cables on this bridge (or any of their components) been repaired? If Yes Explain
20) Whether the strands or cables are replaceable as per design

B. Record of distresses noticed during Inspection

21) Have any corrosion been noted in Stay cable

22) Have any of the cables subjected to rain-wind-induced oscillation?

23) Has cracking or any distress of the outer Barrier (stay pipe) or its connections been noticed ? Yes/No

24) If Yes, give complete details

a) Any longitudinal cracking of sheathing?
b) Any transverse cracking of sheathing?
c) Any Excessive bulging of sheathing?
d) Any damage of sheathing at the connection of damper or cross-ties?

25) Has cracking or misalignment of the guide pipes been noted?

26) If there is protective tape wrapped around the cable sheathing, whether there is any deterioration of the tape? (tear, crack or delamination)

27) Have any problems associated with neoprene boots been noted?

28) Any cable sag noticed?

29) Identification of damage if any to connections between anchorage pipes and cable sheathing (outer barrier/stay pipe).

30) Inspection for damage, loosening, lack of water tightness, and any sign of deteriorations along the full length of MTE and anchorages.

31) Inspection for damage or dislocation of neoprene rings* and keeper rings*, if applicable.

Note:

*Neoprene ring- A stay cable is subjected to lateral movements as a result of vibrations. These vibrations create bending stresses at the two ends of the cable, thus increasing the potential for fatigue. To address this issue, neoprene rubber "washers," "rings," or "donuts" are typically placed around the HDPE pipe within the guide pipe (anchor pipe) near the ends of the cables.

**Keeper rings are steel rings that typically hold the washers in plan. Failure of keeper ring can lead to dislocation and misalignment of neoprene ring, For more details, refer NCHRP Synthesis 353 "Inspection and Maintenance od Stay Cable Systems", TRB [16].

32) Identification of gaps between the neoprene rings and the sheathing.

33) Examination of sheathing surface inside the guide pipe through a boroscope or other means, looking for damage or deformation to the sheathing near the anchorage.

34) Review cracking or damage to guide pipes or evidence of the impact of cable components on guide pipes.

35) Examination of surface conditions on the visible anchorage components including ring nuts, end caps, and bearing plates.

36) Examination of visible parts of saddles for damage, corrosion, and cracking, if applicable.

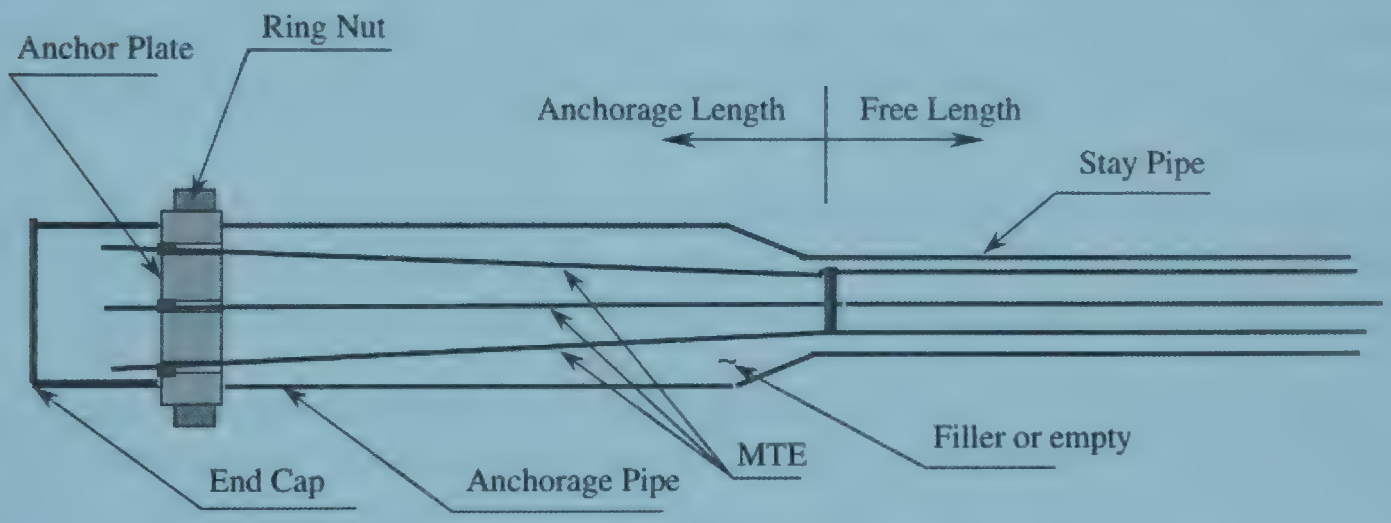
37) Review of evidence of moisture or fillers (such as grease) exiting the anchorage components. If there is an access port at the end cap (ideally at the lowest point), it can be opened and examined for moisture or moisture-contaminated grease

38) After the removal of the end caps on the sockets, visually inspect the anchorage plate and anchorage devices and to see if there is moisture or corrosion inside.

39) Inspection of the cross- tie cables for sagging (losing their force and need to be retensioned).

40) Inspection of damage or cracking on components of cross tie cables. Evidence of fretting and fatigue, especially at connections, are of particular interest.

41) Examination of dampers, if any, as per recommendations of manufacturer



Annexure A7

(Pertaining to Chapter 13)

Design Aspects of Structural Health Monitoring System and Safety Evaluation

This Annexure discusses in detail about the important aspects to be considered while designing a SHM, functional requirement and SHM framework and has been referred in **Chapter 13** of this guideline.

A.7.1 Designing a SHM

The following important aspects shall be considered while designing a SHM

i) When designing a structural health monitoring system, it is fundamentally important to design as an integrated system, which has all data flowing to a single database and presented through a single user interface.

ii) The integration between the different sensing technologies can be simultaneously installed on the structure, for example, electrical strain gauges, fiber optic sensors, vibrating wire sensors, weather stations and corrosion sensors, can be achieved at several levels

iii) Different sensors can be connected to the same data acquisition system or otherwise, several data-loggers can report to a single data management system, which is generally a personal computer installed either on site or at a remote location.

iv) Even though many of the sensors and systems provide their own software for data management and presentation, one of the goals of the SHM design shall provide a single integrated interface that does not require the end user to learn and interact with different user interfaces.

A.7.2 Functional requirements

There are several functional requirements that must be considered when designing SHM system:

i) The system must implement advanced techniques, good performance, long term stability, and economic value rationality.

ii) The system must have the capability to transmit data, process view, archive document and share long distance information.

iii) The system must be able to collect data synchronously, real-time, long term, and hierarchically.

iv) The system must have capability to assess, control, and calibrates itself.

v) The system must be able to identify damages and evaluate structural health.

vi) The system must be reusable and upgradable

A.7.3 SHM Framework

A Structural Health Monitoring (SHM) System is composed of the following modules:

1) Sensory System
2) Data acquisition and transmission system
3) Data processing and control system
4) Structural health evaluation system
5) Structural health data management system, and
6) Inspection and maintenance system (only if SHM is integrated with inspection and inspection)

Modules 1 and 2 are sensors, data loggers and cabling networks for signal collection, processing and transmission; Modules 3, 4 and 5 are computer systems assigned with different functions of system control and system operation. Module 6 has the purpose of performing the inspection and minor maintenance works for modules 1 and 2.

Sensors provide the basic data for a structural health detection and monitoring system. Their performance directly determines ultimate success of the detection and monitoring methods. Sensors need to record the structural mechanical conditions, and directly transform the measured parameters like strain, displacement, acceleration into signals for output. They must sense the variation of external environmental conditions, which needs sufficient reliability, sensibility and high reaction velocity to reflect the external information promptly and accurately.

Usually, the selection of sensors takes the following factors into consideration: the type, precision, resolution, frequency response and dynamic range; the distribution positions, the influence degree of the surrounding dynamic environment and measurement noise.

For the design of sensor modules and sensing technologies for cable stayed bridges, three aspects should be considered: the parameter to be monitored, the sensor type, and the positioning of the installed sensors.

For the selection of sensor systems, the following aspects should be taken into account:

(i)operational bandwidth; (ii)magnitude and frequency response over that bandwidth; (iii)sensitivity and accuracy; (iv)power supply requirements; (v)physical characteristics (dimensions, weight and material) (vi) environmental operating conditions such as temperature ranges; type of output signal; any signal conditioning requirements; and costs.

The commonly used sensor types and requirements are listed in **Table A7.1**.

Table A7.1 Typical Sensor Types and Requirements

Monitoring Category	Specific Parameter/sensor	Sensor requirement Accuracy, range, resolution, sensitivity
Structural global response monitoring	Acceleration/accelerometer	Large span bridge with low-natural frequency: Frequency response range: 0 ~ 100 Hz Range ≥ ±2 g; transverse sensitivity < 5%
Structural local response monitoring	Strain measurement/ Strain guages	30% full range < static strain measurement < 80% full range Range of dynamic strain gage ≥ 2 × maximum predicted value
	displacement Measurement using displacement sensors	LVDT with measuring ranges between ±1 mm and ±50 mm and a quasi infinite resolution within a temperature range between –20°C and +120°C
	Temperature measurement/ Surface mounted vibrating wire guage	Structural surface temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C ; Accuracy ≥ ±0.2°C ; resolution ≥ ±0.1°C
Environment monitoring	Wind speed and wind direction monitoring (3 cup Anemometer)	Range ≥ design wind speed of its installation height
	Wind pressure measurement	−1,000 Pa < range < 1,000 Pa Measurement error ± 2Pa
	Atmospheric Temperature monitoring	Atmospheric temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C; Accuracy ≥ ±0.5°C ; resolution ≥ ±0.1°C
	Humidity measurement	Range: 0–100% RH; accuracy ≥ 3% RH

Data Section

The data section is divided into four parts: collection, transmission, processing, and management. **Table A7.2** deals with the importance aspects to be considered with respect to data related processes.

Table A7.2 Data Requirements

Data	Aspects/parameters	Requirements
Data collection	Selection of equipment	Charge signal: charge amplifier Digital signal: RS485 or compatible with sensor Data collection Analog signal: 4–20 mA and +5 V
	Sampling frequency	Load and environment monitoring: minimum value - Pier acceleration: 50 Hz; wind pressure: 20 Hz; earthquake: 50 Hz; temperature: 1/600 Hz; humidity: 1/600 Hz; rainfall: 1/60 Hz
	Time synchronization Accuracy	Same type of monitoring variables: <0.1 ms Different same type of monitoring variables: <1 ms
Data transmission	Transmission distance	Relatively short and without strong electromagnetic interference: analog signal Far or strong electromagnetic interference: digital signal Up to a few kilometers or even tens of kilometers: optical fiber transmission
Data processing	Pre-processing	Digital filtering, denoising, intercepting
	Post-processing	Specialized design
Data management	Functional requirement	quick display, efficient storage, report generation, and data archiving

A 7.3 Structural Safety Evaluation Requirements

The monitored parameters (load, environment, structural global parameters) can be used for (i) Early Warning and (ii) Safety evaluation of bridge which are discussed below.

Early Warnings

When the structural health monitoring system reports an abnormal situation (as discussed in **Table A.7.3**) to the concerned department, when the global monitoring data exceeds the threshold or limiting value. The threshold value is set according to different levels of danger to the bridge operating environment and structural components

Table A.7.3 SHM data for Early Warning

Safety Warning	Yellow and red warning	(i)Total weight of the vehicle or axle weight > design vehicle load × 1.5 → yellow (>design vehicle load x 2.0 → red) (ii)Maximum average wind speed > design wind speed x 0.8 → yellow (>design wind speed → red) (iii)Maximum/minimum temperature, maximum temperature difference, and maximum temperature gradient > design value → yellow (iv)Displacement > design value x 0.8 → yellow,(>design value or more than 10 yellow warnings appear within a month → red

Safety Evaluations

There are two levels of structural safety evaluation- First level and Second level. Based on the analysis of structural health data collected **Table 11.6** first-level safety evaluation shall be carried out.

In the first level safety evaluation, the eigenvalues of the structure are calculated using the monitoring data, and compared with the predetermined values. If the measured and computed vibration parameters are close, then the bridge is structurally safe. If the measured frequency values are lower than the computed values, then it may indicate a stiffness degradation and thereby indicating the need to perform second-level safety evaluation. Also, if the monitored global response parameters as indicated in **Table A7.4** are abnormal, second-level safety evaluation should be conducted.

Second-Level safety evaluation is a structural safety assessment method in which the corrected structural finite element model, monitored load, and design load are used for structural re-analysis, ultimate load carrying/bearing capacity analysis, and the evaluation of both the bridge structural safety state and the estimated safety reserve.

If the structural local response is found abnormal, special investigations /assessment should be performed. Some of the situations under which special investigations are required are as follows- (i) Measured maximum average wind speed is greater than the design wind speed (ii) barge/ship impact of the bridge (iii) bridge is flooded.

If bridge damage is found during the special assessment, the second-level safety evaluation should be carried out based on the monitored data and special investigations results.

Table A7.4 Global Response Parameters

Structural Health Assessment	**Type of Monitoring**	**Structural performance/ load & environmental Parameters**
Data Analysis	Structural global response monitoring	(a) Acceleration: absolute maximum, maximum root mean square value (b) Deformation: mean, absolute maximum (c) Displacement: maximum, cumulative displacement (d) Modal parameters: frequency, mode, and damping ratio
	Structural Local Response Monitoring	Strain: mean, maximum, minimum, stress amplitude maximum; support reaction time course: mean, maximum/minimum, and maximum change
	Load and environment	(a) Vehicle load parameters: bridge traffic, vehicle type, axle weight, gross weight, speed, and overload ratio (b) Wind parameters: speed, direction (c) Earthquake data: acceleration peak, velocity peak, duration, spectrum, and response spectrum (d) Temperature data: maximum/minimum temperature, the maximum temperature gradient of the section (e) Humidity data: maximum internal/external humidity of the component, mean value, and time exceeding the limit (f) Rainfall data: maximum rainfall per hour, cumulative rainfall

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